SECTION 6 – SEISMIC DESIGN

APPENDICES:

6.1 INTRODUCTION

6.1.1 Overview

The purpose of the Seismic Design Criteria is to establish a basis for the seismic design and performance of UDOT structures. Specific design criteria have been established to achieve this goal, which govern UDOT structure designs. Seismic Design Criteria (SDC) is defined herein for the design of Utah Department of Transportation (UDOT) structures. Design all new bridges in accordance with MCEER/ATC-49, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, as modified by these criteria. All structure designs will be evaluated using these criteria. Structures not meeting the minimum performance criteria set forth in this document are not acceptable and will be subject to more extensive evaluation.

Use the SDC for the seismic design of structures. The SDC follows well-established and accepted principals of performance-based design as documented in several design guidelines and codes.

6.1.2 Background

The approaches presented have been used by advanced transportation agencies in seismic regions such as the California Department of Transportation (Caltrans), and have been the subjects of extensive large-scale testing and peer reviews. The criteria are consistent with MCEER/ATC-49 and clarify the requirements based on UDOT policies, practices, commonly used structural systems, seismic hazards and performance goals. MCEER/ATC-49 was developed as a guide specification for AASHTO LRFD based on NCHRP Project 12-49. The objective of the project was development of seismic design provisions that reflect the latest design philosophies and design approaches resulting in highway bridges with a high level of seismic performance. MCEER was adopted by UDOT in 2003 to provide a comprehensive guide for seismic design.

The Maximum Considered Earthquake (MCE) is defined as the earthquake response corresponding to 2% Probability of Exceedance (PE) in 50 years, equivalent to the MCEER (3% Probability of Exceedance (PE) in 75 years) requirements. The Expected Earthquake (EE) of 10% PE in 50 years is also considered to provide immediate service level functionality after the EE earthquake.

6.1.3 Scope

These criteria apply to the design of bridges. Criteria are presented using a performance based approach. The three performance levels are Operational, Repairable and Life Safety. Performance levels are defined by the expected damage, service and functionality after design earthquakes and depend on the bridge category. These criteria clarify, amend, and add to MCEER/ATC-49 requirements for the design of UDOT bridges. As the basis for these criteria, MCEER/ATC-49 presents and provides new concepts and modifications to current AASHTO LRFD provisions. Sections of this criteria list related MCEER Sections when applicable.

The seismic design criteria, by reference/revisions to the applicable and adopted sections of MCEER/ATC-49, outline the procedure that shall be followed for seismic evaluation and design of highway bridges in the State. In general the procedure defines:

- Seismic Performance Objectives – defining Life Safety and Operational/repairable performance requirements for different bridges and earthquake events,
- Permissible Earthquake Resisting Systems – identifying capacity-protecting elements with ductile/fuse behavior, accessible for inspection/repair
- Seismic Loading – defining site-specific seismic hazard and load combinations to be used,
- Global Demand Analysis – Performance-based demand evaluations for simple, regular, and/or irregular bridges,
- Component Assessment – Local performance-based evaluation procedures to assess and design components to meet performance objectives.

While MCEER provides a wide range of procedures, options, and design strategies, this document outlines the overall procedure, clarifies use of MCEER guides, and amends/revises the guidelines as necessary to meet UDOT requirements. In all cases in absence of specific requirements within these criteria, MCEER guidelines shall apply.

The criteria are accompanied by a commentary document, expanding/clarifying specific provisions as needed. In addition, an example for a 2-span, prestressed concrete girder bridge with integral abutments and a two-column bent has been developed to illustrate application of the criteria for a typical UDOT bridge type.

Caltrans manuals referenced in the criteria are available for free download at http://www.dot.ca.gov/manuals.htm.)

6.1.4 Definitions and Notations

D	Demand
EE	Expected Earthquake
EQ	Earthquake
ERS	Earthquake Resisting System
F	Force or Shear
f_u	Specified Minimum Tensile Strength
f_{ue}	Expected Tensile Strength
f_y	Specified Steel Minimum Yield Strength
f_{ye}	Expected Steel Yield Strength
f'_c	28 Days Concrete Compressive Strength
f'_{ce}	Expected Concrete Compressive Strength
K_{eff}	Effective Linear Stiffness
k_y	Yield Acceleration
L_P	Plastic Hinge Length
M	Moment
MCE	Maximum Considered Earthquake
M_n, M_P, M_{PO}	Nominal moment, Plastic Moment, Overstrength, Plastic Moment
P	Axial Force
SDAP	Seismic Design and Analysis Procedure
SDR	Seismic Design Requirement
T	Period of Vibration
Vn	Nominal shear capacity
Δ_y	Effective yield Column Displacement
δ , Δ	Displacement
δ_{CA}	Allowable compression joint closing
δ_P, δ_y	Plastic displacement, Yield displacement
δ_{TA}	Allowable tension joint opening
$\varepsilon_{C\text{-}A}$	Allowable Confined Concrete Strain
ε_{sh}	Onset of Strain Hardening
$\varepsilon_{SS\text{-}A}$	Allowable Structural Steel Strain
ε_{ssu}	Ultimate Tensile strain (Structural Steel)
ε_{su}	Ultimate Tensile Strain (Mild Steel)
ε_{su}^R	Ultimate Mild Steel Tensile Strain (reduced by 33%)
ε_y	Nominal Yield Strain
ε_{ye}	Expected Yield Strain
ε_{c0}	Unconfined Concrete Compressive Strain at maximum compressive stress
ε_{cu}	Ultimate Confined Concrete Strain
$\varepsilon_{S\text{-}A}$	Allowable Mild Steel Strain
ε_{sp}	Ultimate Unconfined Compression (Spalling Strain)

Definitions and Notations (continued)

ϕ	Curvature Ductility
ϕ_A	Allowable curvature
ϕ_D	Curvature Demand
ϕ_y	Idealized yield curvature
ϕ_{ye1}	Curvature at initial expected re-bar yield
$\gamma_{E\Theta}$	Live Load Factor for Earthquake Load Combination
λ_{MO}	Plastic moment factor over nominal moment
μ_f	Friction coefficient
θ	Rotation
θ_P	Plastic Rotation

6.2. GENERAL REQUIREMENTS

Seismic design of all new highway bridges shall be in accordance with the MCEER/ATC 49 document (MCEER), titled, "Recommended LRFD guidelines for the seismic design of highway bridges," 2003 release and subsequent updates, as revised by Utah Department of Transportation (UDOT) herein and in future updates. Any deviations from these criteria require the approval of the Deputy Bridge Engineer for Design. The Deputy Bridge Engineer for Design represents "UDOT" in all instances noted in these criteria.

6.2.1 Structure Types

(MCEER 3.1)

Typical highway bridge structure types covered by these criteria include:

- Conventional Slab Bridges
- Steel, Reinforced Concrete, or Prestressed Girder bridges,
- Reinforced or Prestressed Box-Girder bridges, and
- Truss superstructures.

Principles of performance-based seismic evaluation and design, as compiled by MCEER and adopted herein by UDOT, can equally apply for other non-typical structure types, in project-specific seismic design criteria, subject to UDOT approval.

6.2.2 Seismic Performance Objectives

(MCEER 3.2)

Performance objectives for the seismic design of bridges shall meet requirements of this section.

Design all new bridges to achieve the specified Seismic Performance Level. This may require exceeding the minimum requirements specified in this document.

Unless otherwise specified by UDOT on a project-specific basis, seismic design loading shall be based on a 75 year life of the structure, and shall consider the effects of dual level ground motion:

- Maximum Considered Earthquake (MCE) Event, also referred to as Maximum Considered Earthquake (MCE) with 2% probability of exceedance in 50 years, and
- Design level Expected Earthquake (EE) event, also referred to as Expected Earthquake (EE), with 10% probability of exceedance in 50 years.

6.2.2.1 <u>Bridge Classification</u>

The Department classifies all bridge structures on the national and state highway systems as "critical bridges," "essential bridges," or "normal bridges." Critical, Essential and Normal bridges are defined as follows:

Critical bridges – Bridges that must remain open to all traffic immediately after the design earthquake (MCE).

Essential bridges – Bridges that can only be closed to traffic for a limited period of time for repairs after a design level event (MCE). Essential bridges should be open to emergency vehicles and for security/defense purposes immediately after the design earthquake.

Normal bridges – Any bridge not classified as Critical or Essential.

Table 6.2.2.1-1 Design Earthquakes and Seismic Performance Levels

Seismic Performance Levels					
Ground Motion Level	Event	Design Levels	Critical Bridges	Essential Bridges	Normal Bridges
Maximum Considered EQ (MCE)	2% PE in 50 yrs	Performance Objective	Operational	Repairable	Life Safety
		Service	Immediate	Limited	Significant Disruption
		Damage	Minimal	Moderate	Significant
Expected EQ (EE)	10% PE in 50 yrs	Service	Immediate	Immediate	Immediate
		Damage	Minimal	Minimal	Minimal

Table Notes:

1. See Notes 1 through 4 in MCEER/ATC-49 Table 3.2-1.
2. *Repairable Damage:* Damage that can be repaired with a minimum risk of loss of functionality. Inelastic response may occur, resulting in concrete cracking, reinforcement yielding, minor spalling of cover concrete, and minor yielding of structural steel. The extent of damage should be sufficiently limited such that the structure can be restored essentially to its pre-earthquake condition without replacement of reinforcement or replacement of structural members. Repairs should not require closure.
3. Locations that can be expected to require repairs following the MCE for the Repairable Damage Performance Level include the following:
   i. Pier columns in plastic hinge zones (top and/or bottom of columns for multi-column bents, bottom of columns for single-column bents)
   ii. Expansion joints may require plating-over immediately following the earthquake as a temporary repair, and may require replacement as a permanent repair
   iii. Shear keys between the superstructure and substructure (if designed as fuse elements - see Section 6.1.3.1 herein)
   iv. Abutment backwalls
   v. Restrainers (if utilized) may require repairs or adjustment
   vi. Bearings may require repair or replacement

6.2.3 Earthquake Resisting Systems (ERS) / Earthquake Resisting Elements (ERE) (MCEER 3.3)

All bridge structures shall have a defined seismic lateral load path, and permissible Earthquake Resisting Systems (ERS) and Earthquake Resisting Elements (ERE), as described herein. MCEER/ATC-49 Section 3.3.1 and Figure C3.3.1-1a illustrate Permissible Earthquake Restraining Systems and Figure C3.3.1-1b illustrates Permissible Earthquake Restraining Elements. MCEER Figure C3.3.1-2 illustrates permissible ERE's that require owner's approval, in addition to corresponding permissible alternatives for which owner's approval is not required. Modifications to the MCEER permissible ERS' / ERE's specific to UDOT new bridge design are given below and in the Commentary Section C6.2.3.1.

Seismic performance of all bridges per Section 6.2.2 shall define individual component performance requirements to meet Life Safety, Repairable, or Operational, as applicable and specified for each project. Provide a dependable seismic lateral load path that ensures the support of gravity loads at all times, and limits damage to permissible ERS systems and ERE's that are typically accessible for inspection.

When the Repairable Performance Level is specified, design and detail the bridge to accommodate the expected repairs. For example, bearing replacement may require additional jacking stiffeners on the girder, a jacking shelf on the support, etc.

6.2.3.1 <u>Permissible Earthquake Resisting Systems / Earthquake Resisting Elements</u>

Specific ERS as specified here are permitted for UDOT projects:

- Limited ductile plastic hinging of flexure members
- Inelastic response of ductile steel bracing
- Fused components (sacrificial element such as shear keys)
- Bridge bearings (either post-failure sliding of typical bearings, or isolation devices)
- Soil mobilization behind abutments
- Spread footing rocking

Passive mobilization of soil behind the abutments is allowed, provided that the soil is engineered fill and specified to be compacted to 95% density, and that the resistance considers upper bound stress limits. Assume that the upper bound stress limit is reached at 2% of wall height, beyond which no additional resistance is assumed.

Note that while the above systems are allowed for the Life Safety, Repairable and Operational performance objectives (as may be required for MCE or EE), the extent of permitted damages are different to meet the specific performance criteria. The interrelationship between the performance objective and the ERS is given in Table 6.2.3.1-a, below.

Furthermore, the following component behavior may be considered permissible, subject to determination and approval of UDOT on a project-by-project basis:

<u>Potentially Permissible Earthquake Resisting Systems / Earthquake Resisting Elements (Requiring UDOT Approval)</u>

These are generally systems or elements in which inelastic behavior occurs in components/locations that are not readily accessible for inspection

- Sliding of superstructure on failed elastomeric bearings (fuse)
- Sliding of superstructure on failed pot-bearings (fuse)
- Essentially elastic behavior (I.e. limited plasticity) of piles

If Potentially Permissible Earthquake Resisting Systems/ Earthquake Resisting Elements (Requiring UDOT Approval) are utilized in a new bridge design, then the SDAP E (elastic response spectrum method with displacement capacity verification) must be used as a minimum.

UDOT will consider other specific strategies and systems with equivalent performance, subject to review and approval on a project-by-project basis.

Table 6.2.3.1-a Performance Objectives and Earthquake Resisting Systems

Performance Objective	Expected Element Behaviors	Earthquake Resisting System	Abutment Performance	
			Ground Motions Having 10% Probability of Exceedance in 50 Years (EE)	Ground Motions Having 2% Probability of Exceedance in 50 Years (MCE)
Operational	Linear Elastic Nonlinear Elastic	Permissible elements designed to resist all seismic loads within displacement constraints. Elements requiring owner approval should not be used.	No damage. Soil passive mobilization is permissible if $\Delta \leq 0.01H$, where H = height of the abutment backwall	No damage. Soil passive mobilization is permissible if $\Delta \leq 0.02H$, where H = height of the abutment backwall
Repairable	Linear Elastic Nonlinear Elastic	Permissible elements designed to resist all seismic loads within displacement constraints. Elements requiring owner approval are permissible.	Limited damage and soil passive mobilization permissible.	Moderate damage, with any damage that does occur being accessible and readily repairable (within months). Soil passive mobilization is permissible.
Life Safety	Linear Elastic Nonlinear Elastic Nonlinear Inelastic	Permissible elements designed to resist all seismic loads within displacement constraints. Elements requiring owner approval are permissible.	Limited damage and soil passive mobilization permissible.	Significant damage. Soil passive mobilization is permissible.

6.2.4 Seismic Design and Analysis Procedures

(MCEER 3.7)

Seismic Design and Analysis Procedures (SDAP), and Seismic Design Requirements (SDR) for all bridges shall meet the requirements of this section. Design all bridges, including single span bridges, using MCEER SDAP C, D, or E, and SDR 4, 5, or 6, as applicable:

- Single span bridges with one primary mode of vibration in longitudinal and transverse directions (each) may use SDAP C method, using single mode analysis approach.
- Multi-span and Irregular single span bridges will require multi-mode response spectrum analysis:

- SDAP-D : Modified elastic response spectrum
- SDAP-E : Modified elastic response spectrum, with displacement capacity check

6.2.5 Capacity Design

(MCEER 4.8)

Inelastic design strategies which limit the magnitude of seismic loads/demands in the lateral load path of the structure (such as ductile column plastic hinging or bearing/shear key fusing) are in compliance with promoted Capacity Design principles, per MCEER and UDOT guidelines. These predetermined and detailed components act as controlled "weak links", thereby limiting the demands to the adjacent/in-path components (i.e. capacity protected components).

While the extent of permitted response for these components will vary, they can be shown to be effective for the full range of Life Safety to Operational performance objectives. Design the protected components to resist upper bound overstrength capacity of the weak-link elements (such as column overstrength moment, or shear key ultimate capacities).

6.2.6 Seismic Design Strategy Report

The design of each bridge (regardless of Seismic Performance Level) requires the preparation of a Seismic Design Strategy Report (SDSR). Submit a preliminary report with the Situation and Layout approval submittal. Submit an updated report with the Design Midpoint Review submittal. Submit the final Seismic Design Strategy Report with the Final Design Review submittal.

The Seismic Design Strategy Report describes the design strategy for resisting the design seismic event. Include in the report descriptions of the following, as appropriate:

- General structure description
- Bridge Category and Seismic Performance Level
- Description of the seismicity at the bridge site
- Description of the design seismic behavior of the bridge, including Earthquake Resisting Systems (ERS) that will be incorporated into the seismic analysis/design
- Locations of plastic hinging
- Redistribution of forces
- Mobilization of backfills
- Function(s) of bearings
- Description of the expected seismic performance of the bridge, including a listing and description of elements that are anticipated to be damaged
- Expected level of damage for those elements (e.g. abutments, shear keys, columns, foundation and expansion joints)

A sample SDSM is attached for reference in Appendix A of this document.

Following the submission of a Seismic Design Strategy Report to UDOT, a Seismic Strategy Meeting between UDOT and the designer may be held to review the report and the design approach. In general, a Seismic Strategy Meeting will be held for complex bridges and for bridges designed to the Operational and Repairable performance objectives.

6.3. LOADS AND COMBINATIONS

(MCEER 3.5)

For earthquake loading combinations, use the Extreme Event I load combination, as described in this section.

Extreme Event I (Earthquake Load Combination): D+γ_{EQ} LL+WA+FR+EQ+E+(PS)

Where:

D – Dead Load and additional Superimposed Dead Load

(Including DC and DW as noted in MCEER)

LL – Live Load; γ_{EQ} - Live Load Factor

WA – Water Buoyancy and Stream Flow

FR – Friction Force

EQ – Earthquake load combinations, corresponding to MCE or EE Earthquakes

E – Earth Pressure (including EH, EV, ES, DD as noted in MCEER)

PS – Prestress (including creep, shrinkage, aging, construction sequence effects)

Note that load components corresponding to active earth pressure (E) and prestressing (PS) have been added to the MCEER-defined load combination, as noted below.

6.3.1 Load Components

The following describe individual load components to be represented in Extreme Event I load combination.

6.3.1.1 Dead Load

The dead load consists of all self-weight of the structure including but not limited to the girders, columns, pier caps, and shear keys. In addition to self-weight dead load, apply additional superimposed dead load in accordance with UDOT specifications, as well as applicable project-specific criteria. Include in the superimposed dead load the weight of components other than the main structure components. It shall include, but not be limited to, future wearing surface, barriers, walls, walkway, lighting, signal masts, and utility poles.

6.3.1.2 Live Load

Unless otherwise specified for a project, use a Live Load factor ($\gamma_{E\Theta}$) of 0.25, when considering Extreme Event I load combination for MCE or EE events.

UDOT will specify higher factors in the range of 0.5 to maximum of 1.0 for essential or critical bridges, bridges with high daily traffic, and/or high truck traffic on a case-by-case basis.

When considering live load effects, consider the Live Load envelop due to gravity of the live loading to only add and increase component responses (i.e. live load cannot cancel and/or reduce the seismic design demands due to other loads). Generally, Live Load considerations shall seek the worst case scenarios where presence of live load will further increase the demands only. Do not consider inertia effects of Live Load.

6.3.1.3 Water Stream Load

In addition to the stream loading included in Extreme Event I load combination, determine the design level channel bed scour, degradation, and aggradation effects at the site. Unless otherwise specified for the project, consider an upper/lower bound of channel bed variations corresponding to mean discharge design levels, when combined with earthquake loading under Extreme Event I load combinations. The upper/lower bound consideration is required because either limit of channel bed conditions may result in conservative design for some components but not others. A bounded approach will capture the worst case results. It is not necessary to duplicate design and analyses efforts. Simplified evaluations in advance can point to which case will govern the seismic design for all or groups of components.

6.3.1.4 Earthquake Load

The load combination applies to both MCE and EE events.

6.3.1.5 Other Load Components

Consider in the Earthquake Load Combinations structures and/or sources of additional loading in the vicinity of the bridge that may become active during the event of MCE or EE. Consider during the MCE and EE events all potential sources of loading within a distance of 50 ft from the bridge.

6.3.2 Combination of Orthogonal Seismic Effects

(MCEER 3.6)

When modal response analysis is used to determine the maximum forces, use the procedures defined in MCEER as the 100% – 40% Combination Rule to combine orthogonal seismic effects. Do not use the SRSS Combination Rule.

Where time-history analysis is used, input ground motions in the orthogonal directions simultaneously (represented by orthogonal time histories with their respective temporal phases and details) and use the maximum combined forces and moments (i.e., 100%-100% rule, not SRSS or 100% – 40% rule).

6.3.3 Ground Motion

(MCEER 3.4)

Determine design ground motion parameters using the procedures specified herein.

6.3.3.1 Design Spectra Based on General Procedure

Use the three-point method to construct the design response spectra for the MCE and EE (instead of the two-point method specified in MCEER Section 3.4.1). In addition to S_s and S_1, use the Peak Ground Acceleration (PGA) parameter to construct the design spectra. At T=0, the response spectra acceleration shall be equal to F_aPGA (as opposed to $0.40S_{DS}$ as specified in MCEER Section 3.4.1).

For periods less than or equal to T_0, revise the design response spectral acceleration, S_a, (MCEER Equation 3.4.1-3) to:

$$S_a = (S_{DS} - F_aPGA)(T/T_0) + F_aPGA$$

For the MCE earthquake, obtain PGA, S_s, and S_1 (for Site Class B, Soft Rock) from the latest national ground motion hazard results published by USGS.

For the EE earthquake, obtain PGA, S_s and S_1 by linear interpolation from the latest national hazard curves published by USGS.

The USGS Java Ground Motion Parameter Calculator may also be used to obtain PGA, S_S and S_1. It is available at the following USGS website address:

http://earthquake.usgs.gov/research/hazmaps/design/

Select Probabilistic Hazard Curves, input the latitude and longitude of the bridge and input the hazard curve value (either the return period or the probability of exceedance and exposure time of the earthquake).

6.3.3.2 Site Effects

The definitions of site class based on the average parameters of the upper 100 ft of the site profile may not be applicable in deep soil basin conditions where the thickness of the soil overburden is much greater than 100 ft. Soil amplification / deamplification effects can significantly change the spectral values for deep soils sites. These effects may not always be adequately represented by using generic soil amplification factors given in MCEER/ATC 49. Special investigations including site specific response analysis may be warranted for deep Site Class D profiles and all Site Class E and F soil profiles. UDOT has developed additional guidance for performing ground response analyses for these soils. See UDOT Research Report UT-03.19, Bartlett, Steven F., "Ground Response Analyses and Design Spectra for UDOT Bridges on Soft Soil Sites," Jan 2004 for more information.

Do not use average N-values or S_u-values (MCEER Equations 3.4.2.2-2 through 3.4.2.2-4) directly to determine the Site Class. Use site-specific V_s measurements whenever possible. For smaller projects where site-specific response analyses are not planned, V_s estimates can be inferred from nearby, pre-existing V_s data for the corresponding geological unit, or obtained from empirical correlations with SPT N and S_u values. Use the resulting shear wave velocities in MCEER Equation 3.4.2.2-1 to obtain the average shear wave velocity and determine the Site Class. If empirical relations are used, consider the uncertainty in the derived V_s values.

6.3.3.3 Site-Specific Response Analysis and Depth of Controlling Motion

In addition to the situation when Site Class D profiles and Site Class E and F soils are encountered, site-specific site response analysis is also required for cases involving deep foundations where the controlling motion is more appropriately specified at depth rather than near the ground surface.

6.3.3.4 Time Histories

In addition to the requirements specified in MCEER Section 3.4.4, it is also required that when a time history dynamic analysis is used for near-field sites (Distance to active fault < 6.25 miles), the recorded horizontal components of motion selected be representative of a near-field condition and that it be transformed into principal components before making them compatible to the target response spectra. Use the major principal component to represent motion in the fault-normal direction and the minor principal component to represent motion in the fault-parallel direction.

For sites with potential near-field effects (distance to active fault is less than 6.25 miles), select candidate time histories for time domain analyses that include near-field effects, such as directivity pulses. Generation of synthetic time histories is discouraged unless the algorithm used is capable of generating near-field effects, such as large velocity and displacement pulses in the synthetic record.

Refer to MCEER/ATC 49, Article C3.4.4 (Commentary) for guidance on the selection and development of time history ground motions.

6.4. ANALYSIS

(MCEER 5)

Performance-based seismic evaluation of bridge structures shall be conducted in accordance with MCEER requirements, for SDAP C, D, and E, as amended and revised herein.

This section defines individual component modeling to be used in seismic evaluations (global analytical models, as well as local push-over and component evaluation models). Performance-based idealization of structure components is directly related to expected and permitted performance per MCEER definitions and requirements.

6.4.1 General Modeling Requirements

(MCEER 5.3)

When performing global analysis, consider expected material properties, as described in Section 6.4.3, in the analytical modeling of the structure components. Specify individual component behavior in the analysis procedure, using representative “element” idealizations, accounting for geometric offsets of connected components at their representative centers of mass and/or stiffness.

Analytical modeling of bridges with multiple frames will generally include both “global” analysis of the overall structural system and “local” analysis of stand-alone frames or bents. The guidelines for “global” and “local” analyses presented in Sections 5.3 through 5.5 of Caltrans Seismic Design Criteria can be followed.

In addition to Section 5 of MCEER/ATC-49 and Sections 5.3 to 5.5 of Caltrans SDC, Caltrans Memo To Designers 20-4 Attachment A – STRUDL Modeling Guidelines provides an overview of modeling techniques, with an emphasis on elastic response spectrum analysis.

Generally, “spine” models in which the superstructure is condensed to a single line will be adequate for most bridges, including curved bridge with a subtended angle less 90 degrees and bridges with skews less than 45 degrees. For seismic analysis of “regular” highway bridges that have only moderate curvature (maximum subtended angle in plan <30 degrees) and do not have large skews (<20 degrees), the bridge may be modeled as straight provided that the span lengths of the equivalent straight bridge are equal to the arc lengths of the curved bridge.

For bridges that have higher curvature (maximum subtended angle in plan between 30 and 90 degrees), larger skews (between 20 and 45 degrees) or varying roadway widths, the analytical model should explicitly model the actual curvature and skew. Particular attention should be paid to modeling of the bearings, i.e. the direction of bearing releases in the model, and the abutment stiffness (orientation) in the model.

If the torsional response of the superstructure is important, e.g. for curved bridges with a subtended angle in plan >90 degrees, or for bridges with high skew (>45 degrees), then a more detailed representation of the superstructure mass and stiffness is warranted and the actual curvature and skew should be explicitly modeled. In such cases, the global analysis can be performed using a general purpose finite element program with the deck modeled using shell elements (e.g. element SBHQ6 in the GTStrudl program). In this case, the number of elements across the width of the bridge should be enough to ensure that the torsional response is correctly computed – generally six elements are sufficient. The spacing of elements in the longitudinal direction should be such that unfavorable aspect ratios (element length/width) are avoided – generally the aspect ratio should not exceed 3.0.

The computer program SEISAB (Imbsen & Associates, Inc. 1999) may be used for seismic analysis of curved highway bridges with a subtended angle in plan <90 degrees) and skews less than 45 degrees, i.e for bridges that can be analyzed using a “spine” model.

There are many general purpose finite element programs available commercially that are suitable for dynamic analysis of bridges, including (but not limited to):

- GTStrudl (Georgia Institute of Technology Computer Aided Structural Engineering Center)
- LARSA (LARSA, Inc.)
- SAP2000 (Computers & Structures, Inc.)

These general purpose programs can be used for dynamic analysis using finite elements models as well as for dynamic analysis using simpler “spine” models.

These general modeling requirements are summarized in Table 6.4.1-1, below.

Table 6.4.1-1 Modeling Requirements

Degree of Curvature (Subtended Angle)	Skew Angle	Modeling Approach	Computer Programs (not all-inclusive)
< 30º	< 20º	3D Spine Model (May model as straight bridge using span lengths equal to the arc lengths of the actual curved bridge)	SEISAB GTStrudl LARSA SAP 2000
30º < 90º	20º < 45º	3D Spine Model (Model as curved using actual bridge geometry)	SEISAB GTStrudl LARSA SAP 2000
> 90º	> 45º	3D Finite Element Model (Model as curved using actual bridge geometry using shell elements for deck)	GTStrudl LARSA SAP 2000

Nonlinear time history analysis should be used for the final design of bridges using isolation bearings. Nonlinear time history analysis can also be used if it is desired to have a more accurate determination of the expansion joint movement demands. Typically, nonlinear time history analysis will result in smaller movement demands for expansion joints than linear response spectrum analysis (i.e. smaller expansion joints).

6.4.2 Component Modeling

6.4.2.1 Superstructure

(MCEER 5.3.3.3)

For typical cases, simple beam elements can be used to capture stiffness and mass contributions of superstructure components.

6.4.2.2 Substructure Columns

(MCEER 5.3.3.2)

For detailed global modeling and/or local stand-alone pushover analysis of the substructure, specific component idealization is required. These idealizations consider both “Elastic Response” (i.e. when un-reduced elastic response establishes governing design demands) or “Ductile or Fused response” (i.e. Capacity Protection Methods, in which column plastic hinging from local analysis establishes governing design demands, or a fused bearing assembly limits force transfer to the ultimate failure/fusing load). In either case, represent component behavior via the “effective and expected” behavior, as per permitted ERS.

Bents/piers shall be idealized with beam (frame) elements, along with associated translational and rotational end fixities. Ductile columns within global analysis shall be represented with their effective section properties, and within local stand-alone push-over analysis, shall be modeled as effective nonlinear elements limiting their strength, plastic rotation, and stiffness to the allowable limits as defined here, and evaluated via section moment-curvature analysis. Steel columns with proper detailing, can be considered to have permitted ductile behavior. Generally, effective section properties correspond to initial yielding event. For reinforced concrete sections, this effective section property (also referred as cracked section property), corresponds to initial yielding of the longitudinal rebar.

Other types of piers include hollow reinforced concrete piers, steel piers, or steel towers. Use the effective stiffness of the substructure model, whether within the linear or nonlinear range of response, in the dynamic analysis.

Account for the plastic action of the pier elements, if permitted, in local push-over analyses, for the displacement capacity check.

6.4.2.3 Integral Joints and Connections

Damage to integral joints and connections between superstructure and substructure components is not allowed. Joint shear performance for integral reinforced concrete joints, and moment resisting connections for steel members shall meet protected component criteria. Use finite size rigid elements or equivalent methods to account for geometric offsets of the connecting components (when significant), to properly assess state of joint stresses caused by forces and moments at the face of the joints.

6.4.2.4 Bearings

(MCEER 5.3.6, 5.3.7, 15)

For typical bridge bearings, account for rotational and translational constraints and/or freedoms of bearing assembly (bearings, shear keys) in the analysis. Individual bearing elements using spring or constraint elements can be used. It is also acceptable to use a single element representing the effective bearing group constraints and properties.

For isolation devices which typically have non-linear force displacement relations, use the effective linear stiffness of the devices. Represent effective linear stiffness for shear keys as constrained (infinitely strong) stiffness, unless the element is a sacrificial element allowing for sliding behavior of the superstructure.

6.4.2.5 Expansion Joints

(MCEER 5.3.7)

Terminal or intermediate expansion joint discontinues shall adequately account for “opening” and “closure” behavior of the expansion joints. When abutment backfill or adjacent structures can have stiffness and/or mass dynamic interaction with the subject structure, represent in the analysis the effective concentrated stiffness and/or mass in the corresponding degrees of freedom.

In addition, consider in the dynamic analysis opening (tension) and closure (compression) behavior at the expansion joints. Separate ‘Tension’ and ‘Compression’ models may be required to bound the response using linear analysis methods such as response spectrum analysis.

6.4.2.6 Sacrificial (Fused) Bearings and Shear Keys

Failure of bearing assemblies (bearings, shear keys,) can be shown to be limited to repairable damage, and therefore may be acceptable. This behavior also leads to a “Fused” behavior at the interface of the superstructure and substructure, limiting (protecting) attached components. Note that the extent of allowed sliding will be different for the Life Safety (no-collapse) performance objective vs the Operational (serviceable) performance objective.

When such behavior is expected at the bearing assemblies, use the effective linear stiffness of the components (established iteratively) in the analyses. The maximum force transfer upon failure depends on the design and the weak-plane of failure. For friction between steel and/or concrete interfaces, a coefficient of friction of 0.3 to 0.5 has typically been used.

Note that although the force limitation of fused behavior (protection of adjacent components) is an advantage of “fused” bearing assembly, the resulting larger displacements at the superstructure-substructure interface must be checked against the available sliding seat width. Also, note that the protected components must be evaluated for the “Upper Bound Force Demands”, which in the case of “Fused Bearing Assembly” is the ultimate failure load, prior to fusing.

6.4.2.7 Mass Distribution

(MCEER 5.3.2)

For typical structure components modeled as equivalent beam elements, account for mass inertia effects by using the appropriate mass density, area and length of those elements. Include all other sources of mass in the analysis via concentrated translational and or rotational mass, as applicable, placed at the respective centers of mass to geometrically capture representative offsets. Alternatively, equivalent mass density for members explicitly modeled in the analysis may be used to account for added mass (such as that of superimposed dead load).

6.4.2.8 Abutments

(MCEER 5.3.5)

Include in the analysis abutment effects, including pile foundations and backfill passive pressure, by using representative springs and iterations to establish effective linear spring stiffnesses.

6.4.2.9 Foundations

(MCEER 5.3.4)

When foundation flexibility affects elastic displacements by more than 20%, account for its translational and rotational effects in the global analyses via representative springs.

Consider the effects of cracking and P-δ in the flexural stiffness of pile bents and drilled shafts.

To account for the uncertainties and variability in subsurface conditions and soil properties, determine the foundation flexibility by considering reasonable upper-bound and lower-bound soil stiffness for applicable cases in MCEER Table 5.3.4-1 - Definition of Foundation Modeling Method.

For spread footing and pile footing foundations modeled as rigid, the mass of the foundation may be ignored in the analytical model (which may be important in achieving a total contributory mass of 90%). However, estimate the inertial effect of the footing and include it in the demands on the supporting piles for pile footings.

Spring constants for spread footings should be developed using equations given in Tables 8.4.2.1-1, 8.4.2.1-2 and Figure 8.4.2.1-1 of MCEER / ATC 49. These computational methods are appropriate for sites that do not liquefy or lose strength during earthquake loading.

Spring constants for pile foundations should consider the nonlinear properties of the piles in evaluating the lateral response of the piles to lateral loads during a seismic event. Include in the analysis group reduction factors provided in the geotechnical report. Consider liquefaction where applicable during the development of spring constants and capacity values for pile foundations (see Section 6.4.2.10, below).

6.4.2.10 Liquefaction

For sites identified as susceptible to liquefaction, consider both the non-liquefied and liquefied conditions in the model of the foundations and structure. During the development of the soil springs for the liquefied case, use appropriate residual resistance (represented by residual p-y curves or residual modulus of sub-grade reaction values for lateral pile response analysis) that are consistent with the liquefied soil conditions.

The development of residual resistance may follow the recommendations provided by MCEER Appendix D.4.2.1 and the associated Commentary. When residual strength is used to derive the residual p-y curves, the relationship presented in Figure D.2.5-1 may be used. It is recommended that the lower one-third values in the data range shown in the figure be adopted in estimating the residual strength.

6.4.2.11 Lateral Spreading and Flow

When lateral spreading and lateral flow are predicted to occur as a result of the liquefaction, consider the effect of the permanent ground movements on foundations and structures. However, the inertial effect of the structural vibration during the seismic shaking and the permanent ground movement effect need not be considered in a combined analytical model. It is recommended that these effects be considered independently (i.e., de-coupled) and not additive, unless agreed to or directed otherwise by UDOT.

6.4.3 Material Modeling

Use probable material properties for “Mild Steel”, “Concrete”, and “Structural Steel” for the seismic design of highway bridges. “Prestressing Steel” properties shall be based on design properties, and when used in concrete superstructure are not allowed to reach post-elastic limits.

Use probable material properties for analytical modeling and/or component capacity evaluations. In all cases, use material test data, when available, instead of minimum design values. In the absence of material test data, use expected material properties based on specified design properties, as defined in this section, in the analyses and evaluations.

6.4.3.1 Mild Steel

Generally, for seismic design and evaluation of highway bridges the expected mild steel strength f_{ye} as a function of design strength f_y is represented as:

$$f_{ye} = 1.1\ f_y$$

The steel stress-strain model is generally used. Simplified material models (such as linear, perfectly plastic-parabolic) may be used.

Maximum limits of mild steel strains in reinforced concrete columns, caused by column plastic hinging, shall correspond to the plastic rotation limits specified per MCEER Section 8.8.6, for Life Safety and Operational performance levels.

6.4.3.2 Concrete

Use expected concrete strength (f'_{ce}) for seismic evaluation and design, instead of the design value (f'_c). In lieu of more project specific test results, the following relations can be used:

$$f'_{ce} = 1.3\ f'_c$$

Use Mander’s concrete stress strain model for confined and unconfined concrete, as applicable. Other equivalent/proven confined concrete models which have correlated well with test data, may be used.

Ultimate concrete strain is a function of concrete confinement, as defined by Mander's model. Maximum limits of confined concrete strains caused by column plastic hinging strains shall correspond to the plastic rotation limits specified per MCEER Section 8.8.6, for Life Safety and Operational performance levels.

6.4.3.3 Structural Steel

Probable structural steel properties shall be based on material testing when available. In absence of test-verified and approved data, expected material properties using the following relation between expected yield strength f_{ye}, and design yield strength f_y may be used.

$$f_{ye} = 1.1\, f_y$$

Simplified and well-accepted steel material properties (such as linear-perfectly plastic-parabolic) may be used. Other generally accepted steel material models which have correlated well with test data may also be used, subject to justification.

Structural steel strain (primarily used in protected superstructure or exceptional substructure components) shall correspond to the plastic rotation limits specified per MCEER Section 8.7.9, for Life Safety and Operational performance levels.

6.5. DEMANDS

(MCEER 4.7, 4.8, 5.4.2, 5.4.3)

Based on Seismic Design and Analysis procedures explained in Section 6.2.4, component demand evaluations shall meet the requirements as explained herein. Use MCEER analysis procedure for (modified) Elastic Response Spectrum and Displacement Capacity Verifications as amended and revised herein.

6.5.1 Response Modification Factors

For SDAP D and E, use the response modification factor procedure, as defined in MCEER, following the analysis procedure explained herein.

Use the elastic response spectrum analysis to establish the basis of demands, and apply specified reduction factors for design strength of ductile components (such as columns), as well as protected components (such as connections). Furthermore, when required, check the displacement demands at supports using local push-over analyses to assess ductile column plastic hinging demands. Check the strength of all protected components adjoining column plastic hinge ends to meet capacity design requirements of Section 4.8 in MCEER.

6.5.2 Capacity Design Procedure

All SDAP procedures (C, D, and E) require compliance with Capacity Design requirements as specified in MCEER Section 4.8, and revised herein.

The design strength of protected elements shall exceed the maximum strength of the permissible protecting components. Criteria recognize two general categories of protecting components:

- Protection by plastic strength of ductile columns (inelastic design)
- Protection by ultimate strength of sacrificial fusing components (e.g. shear keys)

It should be noted and emphasized that permitted Earthquake Resisting Systems (ERS) can be used to achieve both Operational and Life Safety performance objectives, as long as the extent of their inelastic response (i.e. ductility demand of columns, or sliding of the girders on the seat) are properly limited to satisfy the performance objectives at the component performance level. MCEER provides explicit limits of column plastic hinging rotations for each performance level. Fusing conditions at bearing and abutment levels often result in joint movements, which when limited can meet both Life Safety (i.e. no-collapse), as well as Operational (i.e. service) levels of performance.

6.5.3 Local Push-Over Analysis Procedure

(MCEER 5.4.3)

Determination of governing bent deformation and plastic hinging demands and capacities under horizontal loading, within the Capacity Design Approach/SDAP E procedure, requires Seismic Displacement Capacity Verification as described in Section 5.4.3 of MCEER and amended herein.

Generally, the evaluation procedures for SDAP C, D, and E require determination of elastic demands from response spectrum analysis. For SDAP D and E, MCEER Article 4.7 allows the application of reduction factors to elastic demands to design component strengths. For

SDAP D, a check of the column base shear strength considering P-delta effects is required. SDAP E requires an additional Capacity Design check for distribution of plastic loads through resisting frames, and checking component responses to establish displacement capacity of the frames.

Generally, component response assessments for SDAP C, D, and E require the following steps:

1. Governing horizontal displacement demands at each substructure (Bent/Pier) are determined from dynamic analyses of the structure, accounting for Directional Loads. In addition, unreduced elastic force and moment demands for all components are established.
2. For Capacity Spectrum Design Method (SDAP C) and (modified) Elastic Response Spectrum Method (SDAP D), column overstrength moments shall correspond to permitted reduced elastic demands (i.e prescribed modification factors). Local push-over analyses procedure are not required for SDAP C, and D. Design strength of the protected components can then be developed via equilibrium condition with column plastic overstrength (or other permitted strategies such as ultimate capacity of the bearing shear keys).
3. For (modified) Elastic Response Spectrum Method with Displacement Capacity Verification (SDAP E), in addition to the above, local push-over analysis of the substructure systems shall reach governing displacement demands from global analysis, and inelastic deformation demands of columns are determined and compared to push-over displacement capacities. Note that modification factors for SDAP E elastic demands are different than those for SDAP D, reducing the required design strength of the columns.

6.6. COMPONENT PERFORMANCE AND DESIGN CRITERIA

(MCEER 8)

Section 6.2.4 requires SDAP C, D, or E to be used . Seismic performance and design requirements for bridge components shall meet SDR 4, 5, or 6 requirements, as specified in MCEER, and revised herein.

6.6.1 Response Categories

Elastic seismic design of highway bridges are typically limited to rare cases and/or lower level earthquake loadings. Typically, limited inelastic response of bridge components are permitted, as long as the component response as well as bridge response can meet performance objectives and requirements.

For UDOT bridge structures, limited inelastic response of columns with adequate plastic rotation capacity at column ends (plastic hinges) accessible for inspection and repair are permitted. Plastic hinging of the columns will limit seismic loads to adjacent (protected) components.

Repairable damage to secondary components such as sacrificial bearing assembly (bearings, shear keys) and abutment backwall are also permitted. The resulting force transfer limitation will potentially provide for capacity-protection of adjacent essential components within the seismic load path (i.e. fusing behavior) when it can be shown to:

- Have no adverse effects on structure performance,
- Be easily and economically repairable and/or replaceable,
- Meet performance objectives of Life Safety, or Operational, as applicable..

Typically, superstructure, support seats, and foundation elements are to be “protected” components, and shall remain essentially elastic to meet performance requirements.

It is important to note that all permitted ERS’s, can be used for both limits of performance objectives (Life Safety and Operational), or earthquake loadings (EE or EE). The required limit state for the ERS’s will be different for Life Safety and Operational performance objectives.

6.6.1.1 Capacity Protected Components

(MCEER 8.2.2)

All structures are required to meet capacity protection requirements. This includes structures that are designed to resist the anticipated seismic loads with elastic behavior, as well.

6.6.1.2 Ductile Components

(MCEER 8.2.1)

As one of the permitted ERS’s, MCEER allows for limited plastic hinging of ductile reinforced concrete and/or steel columns in meeting Life Safety or Operational performance objectives.

6.6.1.3 Fuse Components

Key components such as bearing shear keys, or sliding behavior of typical bridge bearings can reduce transfer of lateral loads to the adjoining components, and therefore can have the same Capacity Protection features that controlled column plastic hinging has. Subject to the designer's demonstration of the behavior, accounting for upper and lower bounds of response due to uncertainties associated with sliding behavior at the interface, fused response of bearings and shear keys are permitted when they can be shown to meet the Life Safety or Operational performance objectives (as applicable). Note that accessibility for inspection and repair are critical requirements of any ERS, including approved fuse components.

6.6.1.3.1 Superstructure Shear Keys

Shear keys are typically designed to fuse at the Maximum Considered Earthquake event (MCE). Minimum requirements in these criteria are intended to keep the keys elastic at the lower (more frequent) Expected Earthquake event (EE). Fusing of shear keys is permitted only when supports have ample width to tolerate seismic displacements. When excessive seismic displacements must be prevented, provide and design shear keys as capacity protected elements.

For slender bents, shear keys on top of the bent cap may function elastically at the MCE level. For shear keys at intermediate hinges within a span (when permitted), the designer shall assess the possibility of a shear key fusing mechanism, which is highly dependent on out-of-phase frame movements.

Determine the nominal shear key capacity V_{nk} based on a coefficient of friction considering concrete placed monolithically for the shear key, in accordance with AASHTO LRFD 5.8.4. The overstrength shear key capacity V_{ok} shall be calculated using:

$$V_{ok} = 1.5V_{nk}$$

Use the shear key overstrength capacity in assessing the load path to adjacent members.

For cases where shear keys are needed to achieve a reliable performance at the MCE level (i.e. the shear key element is part of the ERS - see Section 2.3 herein), use non-linear analysis to derive the distribution forces on shear keys affected by out-of-phase motions.

When concrete shear keys will be impacted by relatively thin steel elements such as girder flanges, armor them with sufficiently thick steel plates or angles to distribute the line load over an area of concrete to reduce the bearing stress to an acceptable value.

6.6.1.3.2 Column Shear Key Design

Design column shear keys at hinge locations for the axial and shear forces associated with the column's overstrength moment, M_{po}, including the effects of overturning. Locate the key reinforcement as close to the center of the column as possible to minimize developing a force couple within the key reinforcement. Steel pipe sections may be used in lieu of reinforcing steel to relieve congestion and reduce the moment generated within the key. Any appreciable moment generated by the key reinforcing steel should be considered in applying capacity design principles.

6.6.1.4 Single Span Bridges

(MCEER 8.2.5)

Seismic design forces for single span bridges may be based on MCEER simplified approach. For the seismic design of single span bridges with integral abutments, use single mode analysis procedure to determine design loading for the abutments and piles.

6.6.1.5 Displacement Responses

Generally, seismic design procedures are developed to account for displacement ductility capacity of ductile components, such as columns (i.e. inelastic deformations and plastic rotations), and strength design of protected components to the overstrength moment of adjoined ductile components.

In addition to the above inelastic deformation and strength design requirements, seismic design requirements include checking specific displacement responses, as noted here.

6.6.1.5.1 Seat Width at Expansion Joints

(MCEER 8.3.2)

In the global analysis, consider the effects of joint closing (compression model) and joint opening (tension model). Design seat widths using the governing joint opening displacement demands.

6.6.1.5.2 Displacement Capacity Check

(MCEER 8.3.5)

The Displacement Capacity Verification procedure requires local stand-alone push-over analysis of substructure systems to ensure that displacement capacities associated with limited plastic rotation of columns at plastic hinges exceed displacement demands for the specified performance range.

6.6.1.5.3 P-Δ Requirements

(MCEER 8.3.4)

P-Δ effects, when significant, will reduce column flexure strength and ductility capacity reserve for seismic deformations. This penalizing P-Δ effect shall be assessed per MCEER guidelines.

6.6.1.5.4 Design Displacements

It has been observed (Ref. ATC-32) that, for short-period structures, linear-elastic response models tend to underestimate inelastic displacement amplitudes. In order to account for this phenomenon, multiply the horizontal displacements calculated from the elastic response spectrum analysis by the factor R_d to obtain the design displacements.

$$R_d = [1 - 1/R\,]T^*/T + 1/R \;\geq 1.0$$

where:

R_d = Amplification factor applied to elastic model spectral displacements to obtain design displacements

R = Response modification factor. R shall be taken as the maximum value of R used in design of that frame (i.e. use the maximum value for longitudinal / transverse analysis).

T = Fundamental period of vibration of the bridge/structure as a whole, seconds.

T* = Characteristic ground motion period (in seconds) corresponding to the peak of the energy input spectra. Values of T* are given in Table 6.1.5-1 (adapted from ATC-32).

Use the amplified design displacements as the global displacement demand for inelastic static (pushover) analysis. Where nonlinear inelastic time-history analysis is performed, displacements calculated from the inelastic analysis may be used directly in the design.

Table 6.6.1.5-1 Values of Characteristic Ground Motion Period, T*

	Values of T* (in seconds)											
	M_w = 6.5 ± 0.25				**M_w = 7.25 ± 0.25**				**M_w = 8.0 ± 0.25**			
PGA (**g**)	Class B	Class C	Class D	Class E	Class B	Class C	Class D	Class E	Class B	Class C	Class D	Class E
0.1	0.32	0.45	0.46	0.44	0.41	0.53	0.56	0.56	0.51	0.69	0.71	0.71
0.2	0.37	0.44	0.49	0.64	0.42	0.53	0.55	0.74	0.47	0.61	0.65	0.85
0.3	0.35	0.43	0.50	0.73	0.38	0.51	0.55	0.76	0.48	0.64	0.65	0.98
0.4	0.39	0.47	0.50	0.87	0.42	0.56	0.59	0.93	0.46	0.62	0.66	1.04
0.5	0.37	0.46	0.50	-	0.42	0.53	0.62	-	0.45	0.59	0.70	-
0.6	0.35	0.44	0.50	-	0.43	0.54	0.64	-	0.46	0.60	0.76	-
0.7	-	-	-	-	0.50	0.66	0.76	-	0.54	0.71	0.80	-

Note: M_w is the design earthquake moment magnitude.

The designer's geotechnical engineer should determine the soil site class. In lieu of more definite information, the soil site class may be determined base on the mean shear wave velocity over the top 30 m (100 ft) of the ground, as listed in MCEER/ATC-49 Table 3.4.2-1.

6.6.2 Superstructure

(MCEER 8.11)

Superstructure components are among the protected components expected to elastically resist seismic loads during the design level earthquake (MCE).

6.6.2.1 <u>Precast Girders</u>

If continuity of the bottom steel is not required for the longitudinal pushover analysis of the bridge, such steel need not be placed for vertical acceleration at the bent. The required mild reinforcement in the girder bottom to resist positive moment shall be placed during casting of the precast girders while the required top mild steel shall be made continuous and positioned in the top slab.

6.6.2.2 <u>Structural Steel Components</u>

6.6.2.2.1 General

Demonstrate that a clear, straightforward load path exists within the superstructure, through the bearings or connections to the substructure, within the substructure, and ultimately to the foundation. Design all components and connections to be capable of resisting the imposed seismic load effects consistent with the chosen load path.

Accommodate the low of forces in the prescribed load path through all affected components and their connections including, but not limited to, flanges and webs of main beams or girders, cross-frames, steel-to-steel connections, slab-to-steel interfaces, and all components of the bearing assembly from bottom flange interface through the anchorage of anchor bolts or similar devices in the substructure. Design the substructure to transmit the imposed force effects into the soils beneath the foundations.

Include in the analysis and design of end diaphragms and cross-frames the horizontal supports at an appropriate number of bearings, consistent with Articles 6.6.4.1 and 6.6.4.2.

The following requirements apply to bridges with either:

(1) a concrete deck that can provide horizontal diaphragm action, or

(2) a horizontal bracing system in the plane of the top flange, which in effect provides diaphragm action.

Establish a load path to transmit the inertial loads to the foundation based on the stiffness characteristics of the deck, diaphragms, cross-frames, and lateral bracing. Unless a more refined analysis is made, assume an approximate load path as follows:

(1) Assume the seismic inertia loads in the deck to be transmitted directly to the bearings through end diaphragms or cross-frames.

(2) Utilize assumed structural actions analogous to those used for the analysis of wind loadings to develop and analyze the load path through the deck or through the top lateral bracing, if present.

6.6.2.2.2 Shear Connectors

Provide shear connectors on the flanges of girders, end cross frames or diaphragms to transfer seismic loads from the concrete deck to the abutments or intermediate supports.

For the transverse seismic load, the effective shear connectors are taken as those located on the flanges of girders, end cross frames or diaphragms that are no further than $9t_w$ on each side of the outer projecting elements of the bearing stiffener group.

For the longitudinal seismic load, the effective shear connectors are taken as all those located on the girder flange within the tributary span length of the support.

The seismic load at columns/bents should be taken as the smaller of the following:

(1) The overstrength shear of the columns / bents,

(2) 1.3 times the capacity of the bracing systems if they are considered as ductile seismic resisting systems.

The seismic load at abutments should be taken as the smaller of the following:

(1) The overstrength shear of the shear keys,

(2) 1.3 times the capacity of the bracing systems if they are considered as ductile seismic resisting systems.

Nominal strength of the shear connectors is in accordance with Article 6.10.10 of the *AASHTO LRFD Bridge Design Specifications*.

6.6.2.2.3 Superstructure End Cross-Frames

For SDAP B, C, or D a single angle bracing may be used for the diagonal member of the end cross-frame. As this practice is typical and favored for ease of construction, the design process for a single angle bracing shall follow AISC stand alone document on "LRFD Design Specification for Single-Angle Members". This document is included in Appendix F of the AASHTO LRFD Guide Specification.

For SDAP D, double angles with stitches may be used as members of the end diaphragm ERS. Members with stitches shall follow the design process included in the AISC LRFD Specifications Chapter E on compact and non-compact prismatic members subject to axial compression through the centroidal axis.

6.6.3 Joint and Connections

Superstructure and substructure connections are among protected components, and shall ensure a dependable seismic load path providing stable seating support for the superstructure. Design them for column overstrength moment or ultimate capacity of fused components, as permitted and used in design.

6.6.3.1 Integral Concrete Joints

(MCEER 8.8.4)

Design integral joints, including column-cap, column-superstructure, column-footing, and pile/shaft-footing joints, as protected components. Design integral joints to transfer column overstrength moment to bent and pier caps, superstructure, or foundations.

6.6.3.2 Steel Member Connection

Design steel member connections, at all times, as protected components relative to the member end capacities they are connected to. Connection design shall follow principles of Capacity Design procedures.

6.6.3.3 Approach Slab Connection

Connect bridge approach slabs to the superstructure and/or abutment backwalls using fully developed reinforcing steel dowels. Design the dowels to meet the required seismic performance level.

6.6.4 Special Devices

6.6.4.1 Typical Bridge Bearings

If bearings are designed to transfer seismic loads between superstructure and substructure, demands shall not exceed the capacity of the bearing assembly. Demands shall be based on analytical idealization of bearings in the global analysis corresponding to the established behavior (pinned, rigid, or fusing), as permitted. Fused behavior of the typical bearings and assemblies (such as shear keys or anchor bolts) are permitted when shown to meet the performance objectives for the project.

In addition to the above requirements, for the Operational Performance Level, design expansion bearings according to the following criteria:

- Design the expansion bearings to remain functional for the Expected Earthquake (EE), e.g. bearings shall not exceed their design travel limits.
- For the Maximum Credible Earthquake (MCE), expansion bearings may exceed their design travel limits but cannot fall off the bearing seat.

These requirements are addressed with the following technical criteria:

$MR = \Delta T/2 + 1.20\Delta EQ_{EE}$

$\text{Seat Width} = 1.20\Delta EQ_{MCE}/2$

where:

MR = Movement rating, defined as the total anticipated expansion joint movement from the widest opening to the narrowest opening, rounded up to the nearest ½ inch.

ΔEQ_{EE} = Design longitudinal displacement for the EE event (including amplification factor R_d (see Section 6.1.5.4, Design Displacements). This is the total anticipated (two-direction) earthquake displacement.

ΔEQ_{MCE} = Design longitudinal displacement for the MCE event (including amplification factor R_d (see Section 6.1.5.4, Design Displacements). This is the total anticipated (two-direction) earthquake displacement.

A = Expansion joint gap size at the time the joint is set, defined as the clear distance between the inside faces of the adjacent structure spans, rounded up to the nearest ¼ inch.

In addition to meeting the requirements of the AASHTO LRFD Bridge Design Specifications, design bearings in accordance with the following guidelines (Source: AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (Draft), Section 7.9 - Fixed and Expansion Bearings).

These provisions apply to pin bearings, roller bearings, rocker bearings, bronze or copper-alloy sliding bearings, elastomeric bearings, spherical bearings, pot bearings and disc bearings in common slab-on-steel girder bridges. Curved bridges, seismic isolation-type bearings, and structural fuse bearings are not covered by this section.

6.6.4.1.2 Design Criteria

The selection of seismic design of bearings is related to the strength and stiffness characteristics of both the superstructure and the substructure.

Design bearings consistent with the intended seismic design strategy and the response of the whole bridge system.

Rigid-type bearings are assumed not to move in restrained directions. Therefore, assume that the seismic forces from the superstructure are transmitted through diaphragms or cross frames and their connections to the bearings, and then to the substructure without reduction due to local inelastic action along that load path.

Deformable-type bearings having less than full rigidity in the restrained directions but not specifically designed as base isolators or fuses, have demonstrated a reduction in force transmission and may be used in seismic applications. The reduced force transmitted through the bearing shall not be less than 0.4 times the bearing dead load reaction.

6.6.4.1.3 Design and Detail Requirements

Consider the impact on the lateral load path due to unequal participation of bearings considering connection tolerances, unintended misalignments, the capacity of individual bearings, and skew effects.

Do not use roller bearings or rocker bearings in new bridge construction. Design expansion bearings and their supports in such a manner that the structure can undergo movements in the unrestrained direction not less than the seismic displacements determined from analysis without collapse. Provide adequate support length for fixed bearings.

In their restrained directions, design and detail bearings to engage at essentially the same movement in each direction.

Neglect the frictional resistance of the bearing interface sliding-surfaces when it contributes to resisting seismic loads. Conversely, calculate the frictional resistance conservatively (i.e., overestimate) when the friction resistance results in the application of greater force effects to the structural components.

Provide elastomeric expansion bearings with anchorage to adequately resist the seismically induced horizontal forces in excess of those accommodated by shear in the pad. Make the sole plate and base plate wider to accommodate the anchor bolts. Inserts through the elastomer are not permitted. Design the anchor bolts for the combined effect of bending and shear for seismic loads. Provide elastomeric fixed bearings with horizontal restraint adequate for the full horizontal load.

Evaluate spherical bearings for component and connection strength and bearing stability.

Pot and disc bearings should not be used for seismic applications where significant vertical acceleration is present. Where the use of pot and disc bearings is unavoidable, provide an independent seismically resistant anchorage system.

6.6.4.1.4 Bearing Anchorage

Provide sufficient reinforcement around the anchor bolts to develop the horizontal forces and anchor them into the mass of the substructure unit. Provide sufficient shear friction capacity to prevent failure at potential crack surfaces next to the bearing anchorage.

6.6.4.2 <u>Seismic Isolation</u>

(MCEER 15)

Use of seismic isolation is permitted, provided the behavior, modeling, and range of response meet MCEER requirements for modeling, analysis, upper and lower bound variation considerations, design and prototype testing programs.

6.6.4.3 Typical Expansion Joints

For the Operational Performance Level, in addition to meeting the requirements of the AASHTO LRFD Specifications, design the expansion joints and expansion joint gaps according to the following criteria:

- Design the expansion Joints to remain functional for the Expected Earthquake (EE).
- For the Maximum Credible Earthquake (MCE), expansion joints may fail in tension or compression.
- Design the structural gaps (gap between the edges of the superstructure) to remain open during both the EE and the MCE.

These requirements are addressed with the following technical criteria:

$MR = \Delta T/2 + 1.20\Delta EQ_{EE}$

$A = 1.20\Delta EQ_{EE} - \Delta PS + \Delta T/4$

Structural Gap = $\Delta EQ_{MCE}/2$

where:

MR = Movement rating, defined as the total anticipated expansion joint movement from the widest opening to the narrowest opening, rounded up to the nearest ½ inch.

A = Expansion joint gap size at the time the joint is set, defined as the clear distance between the inside faces of the adjacent structure spans, rounded up to the nearest ¼ inch.

ΔEQ_{EE} = Design longitudinal displacement for the EE event (including amplification factor R_d (see Section 6.6.1.5.4, Design Displacements). This is the total anticipated (two-direction) earthquake displacement.

ΔEQ_{MCE} = Design longitudinal displacement for the MCE event (including amplification factor R_d (see Section 6.6.1.5.4, Design Displacements). This is the total anticipated (two-direction) earthquake displacement.

ΔT = Total design thermal displacement range.

ΔPS = Prestress shortening that is expected to occur after the expansion joint is set.

The above requirements for expansion joints are intended to minimize damage to the expansion joint during the EE event and to avoid excessive damage to the expansion joint area due to banging during the MCE event.

The above requirements need not apply at locations where relative movement between the superstructure and substructure are restricted by shear restriction devices (e.g. shear pins surrounded by a compressible material). For this case, design the expansion joints to accommodate the maximum movement permitted by the shear restriction devices.

6.6.5 Substructure

Evaluate substructure component performance in conjunction with the design approach selected:

- For elastic design, all components shall elastically meet governing demands from dynamic analysis.
- For capacity protected design, protected components shall have adequate elastic strength to resist limiting forces and moments.

In addition, via simplified equilibrium considerations or local stand-alone substructure and/or frame push-over analyses, all protected substructure components shall demonstrate elastic design capacities in excess of either of the two following capacity protection events, as applicable (i.e. selected ERS):

- Column yielding, or
- Brittle failure of a sacrificial “Fuse” Elements (like shear keys)

6.6.5.1 Sub-Structure Columns and Bents/Piers

(MCEER 8.7.4, 8.7.9, 8.8.2, 8.8.6)

Reinforced concrete columns may be designed to respond to seismic loading within the elastic range, or within limited post-elastic range. Steel columns, can similarly be shown to have adequate ductile detailing as allowed for confined reinforced concrete sections. Limit inelastic rotations of ductile column components to meet component requirements for the specified performance objectives.

Hollow piers (reinforced concrete or steel piers) may also be detailed such that the substructure can have adequate inelastic rotational capacity. These cases will be evaluated on a case-by-case basis. Demonstrate equivalence to MCEER plastic rotation limits for ductile reinforced concrete and steel members for Life Safety and Operational performance objectives with test-proven behavior.

Solid reinforced concrete bents are the most common substructure type, ideal to be designed to experience limited inelastic response, and by design perform as a weak link and limit force transfer to adjoined protected components.

Historically, precast girders lacked a direct positive moment connection between the girders and the cap beam, which could potentially degrade to a pinned connection in the longitudinal direction under seismic demands. Therefore, to provide stability under longitudinal seismic demands, columns shall be fixed at the base.

6.6.5.1.1 Solid Reinforced Concrete Columns

(MCEER 8.8.6, 8.8.2.3)

Confined reinforced concrete columns can reach higher plastic deformation capacities, under seismic loading, while limiting force transfer to the adjacent components to their overstrength moments. Inelastic column deformations are concentrated at fixed column ends, within the Plastic Hinging Zone, as described in Section 8.8.6 of MCEER.

For inelastic demands, design reinforced concrete column shear as a protected component to meet column overstrength requirements, and to avoid brittle shear failure. Meet the requirements of MCEER Section 8.8.2.3 for shear design.

Reinforced concrete bent walls can be detailed to meet column plastic hinging requirements in the weak direction, while remaining elastic in the strong direction.

MCEER defines column plastic rotation limits for Life Safety and Operational performance, per Section 8.8.6 requirements. In lieu of the simplified plastic rotation values, column inelastic deformation capacities can be based on column section moment-curvature relationship where allowable concrete and/or steel strains determine plastic rotation limits. Meet MCEER Section 8.3.4 P-Δ requirements for pier displacement demands.

Design column flares following the guidelines in Caltrans SDC, Section 7.6.5 – Column Flares. For the isolation gap, use a minimum thickness of the larger of 2 inches or 1.5 times the calculated plastic rotation demand from the pushover analysis times the distance from the center of the column to the extreme edge of the flare.

6.6.5.2 Substructure Cap Member

Bent cap members are capacity-protected, and shall have sufficient elastic strength to meet governing shear and flexure demands, per Capacity Design requirements of MCEER.

Meet P-Δ requirements for pier displacement demands in MCEER Section 8.3.4.

6.6.6 Foundation Components

6.6.6.1 Abutments

Design abutments to meet the design requirements in MCEER Section 8.5, as amended and revised herein.

Revise the total passive force in MCEER Equation 8.5.2.2-1 to $P_p = p_pHW$, where W is the wall width.

In the longitudinal direction, calculate the earthquake-induced active earth pressure using horizontal accelerations at least equal to 50% of the site peak ground acceleration (i.e., $PGA_D/2.0$).

For transverse loading when a fusing mechanism is chosen for pile supported foundations, the overstrength capacity of the shear keys shall be less than the combined plastic shear capacity of the piles.

Inelastic behavior of piles at abutments is acceptable only with UDOT's approval. When joint closure at the abutments is considered, backwall failure leading to passive backfill resistance is considered as a local and repairable damage. Backfill spring stiffness properties shall consider upper bound resistance based on soil data. Assume that a horizontal displacement of 2% of the backwall height mobilizes the full capacity of the soil reaching the upper bound resistance force, which then remains constant for higher movements into the abutment. Use an iterative effective stiffness which limits backfill force for higher displacements at abutments.

For abutment backfill contribution, compression-only springs may be used to model the resistance upon (repairable) backwall failure. Use passive soil resistance based on site soil data. When site soil data cannot be obtained, the presumptive passive pressure of 2H/3 ksf in MCEER / ATC 49, articles 7.5.2.2 and 8.5.2.2 may be used. In both cases, the full passive resistance is not achieved until the wall displaces a distance of 2% of the wall height. Determine the effective linear stiffness via iteration.

6.6.6.2 Footings

(MCEER 8.4.2)

As an alternative, the shear modulus (G) used to compute the stiffness values in MCEER Table 8.4.2.1-1 can also be determined by the strain-compatible shear modulus derived from a site-specific site response analysis (such as by SHAKE analysis).

Base the seismic design of spread footings for SDR 4, 5, and 6 on column moments and shears developed using capacity design principles. The responses of the footing to shear force and moment can be treated independently (i.e., de-coupled).

Under the moment load from the MCE, transient yielding at the toe and liftoff at the heel of the footing are acceptable provided that

- The global stability is preserved, and
- Settlements induced by the cyclic loading are acceptable.

Non-triangular stress distributions or greater than 50% liftoff are allowed only if special analysis can demonstrate that soil settlement from cyclic loading does not exceed amounts that result in unacceptable damage to bridge structures, and the results are approved by UDOT. Note that, as is the case with all permitted strategies, the extent of allowed footing behavior for Life Safety or Operational performance objectives are different.

6.6.6.3 Piles/Drilled Shafts

(MCEER 4.9)

Piles and shafts are typically protected components requiring essentially elastic performance under MCE and EE events. In isolated cases, including cases where liquefaction-induced lateral spreading and flow occur, limited plastic hinging of piles and drilled shafts may be permitted, subject to review and approval by UDOT on a project-by-project basis. For such cases, special detailing of plastic hinges zones shall meet requirements of MCEER Section 4.9.

Consider the non-linear properties of piles and drilled shafts in evaluating the lateral response of the piles/drilled shafts.

To determine the distribution of forces (shear and axial force) to the piles for pile supported foundations, simplified elastic analysis using linear distribution forces method can be used provided that

1. the pile cap can be reasonably considered rigid,
2. the pile group reduction effect on lateral response is negligible,
3. the piles are in competent soil, such as in Site Class A, B, C, or D, and
4. standard pile sizes are used (with a nominal dimension of 12.75 inches).

For non-standard size piles, the distribution of forces to the piles and the pile cap may be influenced by the fixity of the pile connection to the pile cap in addition to the overall piles/pile cap flexibility. In soft soils or in situations where pile group reduction effect on lateral pile response is significant, the distribution of axial and shear forces to each individual pile can deviate from results from the simplified elastic analysis. A more refined model that takes into account the pertinent parameters is recommended for establishing a more reliable force distribution.

For a single column/shaft, ensure a stable length by using the lesser length determined by one of the following methods:

1. 1.5 times the stable length achieved by applying lateral forces based on overstrength properties used in determining the tip of the shaft required for lateral stability, and
2. applying a 1.5 multiplier factor on the lateral forces based on overstrength properties used in determining the tip of the shaft required for lateral stability.

In addition, do not exceed the ultimate geotechnical capacity of a single column/shaft foundation in compression and uplift under maximum seismic loads.

6.6.7 Soil Effects

(MCEER Appendix D)

6.6.7.1 Liquefaction Effects

The evaluation of liquefaction, including preliminary screening, and the liquefaction effects and the design requirements on foundations and structures shall be in accordance with MCEER Appendix D – Provisions for Collateral Seismic Hazards and described herein.

With UDOT's approval, it is acceptable to limit analysis and mitigation due to the effects of liquefaction to a depth of 80 ft.

Consider the loss of lateral support for pile/drilled shaft foundation by using appropriate residual resistance. When residual strength is used to derive the residual resistance (e.g., p-y curves), the relationship presented in Figure D.2.5-1 may be used. It is recommended that the lower one-third values in the data range shown in the figure be adopted in estimating the residual strength.

6.6.7.2 Lateral Spreading Effects

Evaluate liquefaction-induced lateral spreading and flow and their effects and design requirements on foundations and structures in accordance with MCEER Appendix D – Provisions for Collateral Seismic Hazards and described herein.

Pinning effects of the pile/drilled shaft foundations (used in providing resistance to lateral spreading and flow) and the forming of plastic hinge in piles/drilled shafts are allowed with UDOT's approval.

It is recommended that the Simplified Newmark Charts presented in MCEER Figures D.2.5-2 and D.2.5-3 be replaced with the latest results (expressed as an equation below) based on the on-going NCHRP 12-70 study as follows:

$$\text{Log}(d) = -1.51 - 0.74\log(k_y/k_{max}) + 3.27\log(1 - k_y/k_{max}) - 0.80\log(PGA_D) + 1.59\log(PGV_D)$$

where:

d: displacement in inches,

PGA_D: design peak ground acceleration in g

PGV_D: design peak ground velocity in in/sec

6.6.8 Structure Discontinuities

6.6.8.1 Expansion Joints

Adequately size expansion joints separating structure frames to prohibit unseating of supported structure under joint opening behavior. Also consider joint closure conditions which couple adjacent frames. Local damages at expansion joints caused by joint closure are typically minor.

Under open joint conditions, individual frame responses are assessed. Under closure conditions (i.e. frames locked at the joints), coupled response of adjacent frames are assessed. For adequacy of available joint seat open joint conditions shall be checked against the requirements of MCEER Section 8.3.2

Joint opening in excess of available seat is prohibited. In joint closure (e.g. locking frames at expansion joints), typical conditions will result in local repairable damage at superstructure interface. The main item of interest for the required joint closure analysis is to allow for distribution of superstructure inertia among substructure piers of varying stiffness. This condition causes the governing loading on piers.

Note that for frames next to abutments, the joint closure case at the abutment expansion joint will represent the worst case movement into the abutment. Joint opening at abutments will represent the worst case to assess adequacy of abutment seat width in providing support for superstructure.

6.6.8.2 Restrainers

An adequate method for designing restrainers at expansion joints is not currently available. Therefore, do not rely on restrainers as the primary means of preventing superstructure unseating in new bridge designs. Instead utilize other means , such as:

- providing adequate seat widths
- providing balanced frame stiffness (see Section 6.7.1.3 herein and Caltrans SDC Section 7.1).

Restrainers may be used as a *secondary* means of preventing superstructure unseating at expansion joints within a span at the ends of simple spans for new bridge design.

Restrainers may be used as a primary means of preventing unseating in the seismic retrofitting of existing bridges. However, even for existing bridges other methods of preventing unseating are preferable, including:

- providing bolsters at girder supports
- eliminating intermediate expansion joints
- replacing expansion bearings with fixed or isolation bearings

Where restrainers are used, they can be designed following the guidelines in Caltrans Memo To Designers 20-3 (May 1994). The capacity of the restrainers shall not exceed the capacity of the superstructure or the connecting elements.

6.7. STRATEGIES AND DETAILS

6.7.1 Seismic Design Strategies

Strategies consist of a defined combination of behaviors that incorporate both elastic and inelastic performance of structural and geotechnical components of the design, resulting in a structure that can be demonstrated to meet the overall level of performance required (either no-collapse, repairable or operational).

6.7.1.1 Earthquake Resisting Systems

The Earthquake Resistant Systems (ERS) noted in Section 6.2.3.1 are applicable and can be used in combination. They are expanded upon as follows:

- Plastic hinging in predetermined locations in columns, with limited ductility for operational performance, if required. Plastic hinges can be utilized as an ERS in the following locations:
  - in columns at the base of column/top of footing interface in single or multi-column bents
  - in columns at the top of column/underside of superstructure interface
  - in drilled shafts for Type I Shafts with limited inelastic deformation. The global displacement ductility demand, μ_d, for a Type I shaft shall be less than or equal to the μ_d for the column supported by the shaft (refer to Caltrans SDC Section 2.2.4 and Figure 2.4).

  Plastic hinging is not permitted in the following locations:
  - in bent cap beams of multi-column bents
  - in the superstructure
  - in footings
  - in drilled shafts for Type II Shafts (refer to Caltrans SDC Section 2.2.4 and Figure 2.4)
- Energy dissipation through inelastic behavior of ductile elastic steel bracing, with adequate connection strength without weakening vertical load capacity.
- Fused components, including shear keys and backwalls that can be repaired easily.
- Bridge bearing damage that can easily be replaced.
- Soil mobilization behind abutments that can be repaired.
- Limited rocking of (spread) footings is permitted, provided elastic response of the footing can be demonstrated.
- Make superstructures continuous, providing for a minimum number of expansion joints to account for temperature and shrinkage superstructure movement.
- No In-span hinges.

6.7.1.2 Earthquake Resisting Systems (ERS) With Permission (See Section 6.2.3.1)

The following ERS may be used with UDOT approval:

- Sliding of fused bearing, as long as it meets performance objectives (i.e. limited for operational)
- Essentially elastic response of piles (provided essentially elastic response of pile group can be illustrated) through either:
  - (a) Essentially elastic stress of the pile.
  - (b) Inelastic response of the soil.

6.7.1.3 Frame Design (See Caltrans Seismic Design Criteria - Section 7.1)

The best way to increase a structure's likelihood of responding to seismic attack in its fundamental mode of vibration is to balance its stiffness and mass distribution. Irregularities in geometry increase the likelihood of complex nonlinear response that cannot be accurately predicted by elastic modeling or plane frame inelastic static modeling.

The designer should attempt to balance frame stiffness and mass to avoid the potential adverse impacts to structural behavior that result from the dynamics associated with such unbalanced frames. Consider the recommendations in Section 7.1 of the Caltrans Seismic Design Criteria (SDC) when establishing the bridge layout and framing configuration during the preliminary phase. The designer should follow Caltrans SDC Section 7.1 as a guide to assess and modify frames such that an optimal arrangement for the site can be achieved. Other key elements in frame configuration that need to be considered:

- Where multiple frame structures are required, design each frame to resist it's own seismic demands
- Balanced frame resistance
- Balanced dynamics

The techniques listed in Caltrans SDC Section 7.1.3 can be considered when necessary to adjust the dynamic characteristics (modal periods) to meet the balanced stiffness recommendations given in Caltrans SDC Sections 7.1.1 and 7.1.2.

6.7.1.4 Pile foundations

- Use a pinned head connection when the pile group foundation is governed by flexure response to column base moments
- Use a fixed head connection when the pile group foundation is governed by lateral loading at the top soil layers.

6.7.2 Design Details

6.7.2.1 Caltrans SDC

Use design details per Chapter 8 of the Caltrans SDC.

6.7.2.2 Column Plastic Hinge Zone

Meet MCEER Section 8.8.2.4 thru 8.8.2.6 for confinement requirements.

6.7.2.3 Ductile Reinforcing

Use low-allow steel deformed bars conforming to ASTM A 706 for substructure elements (bent caps, columns, footings, etc.), including internal bent caps that are expected to either (1) form a plastic hinge, or (2) act as a capacity protection element.

6.7.2.4 Column-Cap and Column-Footing Joint

Continue the column hoop or spiral reinforcement into the cap or footing to confine the joint for principle tension stresses. See MCEER Section 8.8.4.3.4.

6.7.2.5 Integral Concrete Column/Cap Joint Details

Consistent with the MCEER (and AASHTO) specifications, if the principal tensile stress p_t exceeds $3.5\sqrt{f_{ce}}$, include the following details in the design.

- Extend the bent cap 12 inches on each side of the column.
- Extend vertical column bars into the cap to the top bent cap reinforcement in order to fully develop the strut-and-tie mechanism.
- Provide vertical stirrups or ties, equal to 20% of the vertical column reinforcement being anchored into the cap, within D_c of the column centerline (measured from either side of the column or wall).
- Provide horizontal stirrups or ties equal to 10% of the vertical column reinforcement being anchored into the cap in layers vertically spaced at not more than 18 in. on center.
- Provide horizontal side reinforcement equal to 10% of the bent cap longitudinal reinforcement, top or bottom, whichever is less. Maximum permissible spacing is 12 in.
- For integral concrete box girders with bridge bents skewed more than 20^o, hook vertical J-dowels equal to 8% of the vertical column steel around the longitudinal top deck steel within a distance of D_c from the face of column. Alternate hook lengths between 24 and 30 in.
- Provide column confinement steel extending into the cap per MCEER 8.8.4.3.4.

6.7.2.6 Footing Reinforcement

6.7.2.6.1 Provide top and bottom mat reinforcement to develop the overstrength moment (M_{PO}) such that damage under column plastic loading does not extend into the footing or pile cap. Connect top and bottom mats of reinforcement using vertical shear stirrups.

6.7.2.6.2 Provide footings and pile caps with shear reinforcement in the form of vertical stirrups at a spacing of no more than 12 in. on center in both the longitudinal and transverse directions.

6.7.2.6.3 Design column footing and pile footing connections to account for joint shear. See Section 7.7.1.4 of the Caltrans SDC for methodology to check footing joint shear.

6.7.3 Foundation Design

Apply the foundation loading in incremental diagonal directions to produce the maximum pile and footing loads.

6.7.3.1 Loading

In addition to the requirements of MCEER, apply off-normal axis loading (diagonal loading) of M_{PO} to the foundations. Design the piles and foundation for these loads. Determination and application of these loads can be accomplished by following equations 7.30 and 7.31 of the Caltrans SDC, or by applying the column plastic loads using other means such that the maximum loading for a plastic condition is determined for the pile group.

$$\left.\begin{matrix} C_{(i)}^{pile} \\ T_{(i)}^{pile} \end{matrix}\right\} = \frac{P_c}{N} \pm \frac{M_{p_{(y)}}^{col} \times c_{x(i)}}{I_{p.g._{(y)}}} \pm \frac{M_{p_{(x)}}^{col} \times c_{y(i)}}{I_{p.g._{(x)}}} \qquad (7.30)$$

$$I_{p.g._{(y)}} = \sum n \times c_{y(i)}^2 \qquad I_{p.g._{(x)}} = \sum n \times c_{x(i)}^2 \qquad (7.31)$$

Where:

$I_{p.g.}$ = Moment of inertia of the pile group defined by equation 7.31

$M_{p_{(y),(x)}}^{col}$ = The component of the column plastic moment capacity about the X or Y axis

N_p = Total number of piles in the pile group

n = The total number of piles at distance $c_{(i)}$ from the centroid of the pile group

P_c = The total axial load on the pile group including column axial load (dead load+EQ load), footing weight, and overburden soil weight

6.7.3.2 Confinement Check

For fixed pile footing joints, perform a confinement check to limit tensile stresses under joint shear.

6.7.4 Post-Earthquake Repairs

The design of new bridges for the Repairable performance level shall consider access to and repair of elements that are designed as fuse elements (e.g. shear keys and bearings) and other components that are expected to be damaged during the design earthquake (see Section 2.2.1 herein). For bridges with bearings that are designed as fuse elements, provide jacking stiffeners or other means to facilitate resetting/repair of the bearings following an earthquake. Show in the design drawings the proposed jacking scheme, jacking location and jacking loads for bearing repair/replacement operations.

6.8. REFERENCES

6.8.1 Design Standards Codes, and Guidelines

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- MCEER/ATC 49 - Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY, November 2001.
- AASHTO Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, 1996.
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- ACI 318, American Concrete Institute, 1999.
- ASD Manual of Steel Construction, 9th Edition, American Institute of Steel Construction,
- Caltrans Seismic Design Criteria Version 1.4, California Department of Transportation, June 2006.
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- NCHRP 12-70 – Draft Final Report: Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments, 2007.
- ATC-32, Seismic Design Criteria for California Bridges: Provisional Recommendations, Applied Technology Council (ATC), 1996.

6.8.2 Technical References

1. Mander, J.B., Priestley, M.J.N., and Park, R., “Observed Stress-Strain Behavior of Confined Concrete,” Journal of Structures Division, ASCE, August 1988.

2. Dowell, R.K., and Hines, E.M., "Plastic Hinge Length of Reinforced Concrete Bridge Columns," Proceedings of the Third National Seismic Conference and Workshop on Bridges and Buildings, 2002.
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5. Paulay, T., and Priestley, M.J.N., Seismic Design of Reinforced Concrete and Masonry Buildings, New York, NY, 1992.
6. Mander, J.B., Priestley, M.J.N., and Park, R., "Theoretical Stress-Strain Model for Confined Concrete," Journal of Structures Division, ASCE, August 1988.
7. SCDOT Seismic Design Specifications for Highway Bridges, Couth Carolina Department of Transportation, 2001 with October 2002 Interim Revisions.
8. WSDOT Bridge Design manual M 23-50, Washington State Department of Transportation, February 26, 2007.
9. UDOT Research Report UT-03.19, Bartlett, Steven F., "Ground Response Analyses and Design Spectra for UDOT Bridges on Soft Soil Sites," Jan 2004.

APPENDICES

Sample Seismic Design Strategy Report (SDSR)

LEGACY PARKWAY

BRIDGE 37 SEISMIC DESIGN STRATEGY

INTRODUCTION

Seismic Design of the bridges for the Legacy Parkway project is primarily based on AASHTO, amended by project specific design criteria, which addresses issues not specifically accounted for by the AASHTO code.

Geotechnical studies of the site for this structure show that there are layers of soil ranging from 1 meter to 4 meters thick, at depths of approximately 3 to 7 meters below existing ground (bottom of pile cap to 4 meters below bottom of pile cap) that will liquefy and spread laterally to the extent of 1.4 meters to 6.7 meters. UDOT Geotechnical guidelines recommend mitigation in the form of ground modification when such hazard is deemed to occur[1].

For the seismic analysis and evaluation of this structure, it is assumed that such mitigation has taken place. If the mitigation has not taken place, then there is no expectation of potential for serviceability, and in fact the analysis cannot conclude that collapse will not occur.

GENERAL STRUCTURE DESCRIPTION

The structure is 96 m long with two-spans (48.000 m – 48.000 m) of precast concrete I-girders made continuous over the bent cap with a diaphragm. The substructure consists of integral abutments and a double-column bent with non-integral cap. The abutments are supported by a single row of concrete filled steel pipe piles, 460 mm in diameter. There are two fin walls at each abutment in addition to the wing walls. The columns are octagonal with a diameter of 2.500 m and is supported on pile footing, using concrete filled steel pipe piles. The superstructure consists of pre-stressed concrete girders made composite with a concrete deck. Superstructure girders are supported on elastomeric bearings.

LEVEL OF PERFORMANCE

This structure is designed as an “Essential Bridge” under the definition provided by AASHTO (Division 1-A). The seismic performance of the structure is “Serviceability”, following minor repairs, which may be required, after the event of “Design Level Earthquake”. This assumes that remedial measures have been taken to mitigate potential liquefaction at the site.

If liquefaction remedial measures are not undertaken, then the structure is not expected to survive the design event and collapse is possible.

SEISMICITY

The structure is designed for PGA = 0.6g. AASHTO curves for Type III soil are used to account for soil amplification.

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[1] Memo from K.N. Gunalan to Ernie Green, August 14, 2002. Subject; Liquefaction/Lateral Spread Analysis – Shepard Lane Braided Bridges 37 & 38.

SEISMIC BEHAVIOR

Seismic performance and resistance of the two-span structure is primarily provided by:

- Abutment back-wall/backfill resistance in longitudinal direction
- Abutment Fin-wall/backfill resistance in Transverse direction
- Bent-Column Flexure resistance
- Abutment piles in longitudinal and transverse directions
- Continuity of the superstructure over the bent cap

Girder bearing connections result in a pin-fix behavior of column flexure in the bent's longitudinal direction, and fix-fix behavior in the transverse direction. Girders are "Keyed" to the cap to transfer seismic lateral loads to the bent, upon closure of minimal service load shear key gaps.

EXPECTED DAMAGE UNDER DESIGN EVENT

Assuming that liquefaction mitigation measures are put in place, such as jet grouting of the soil, the expected inspection and immediate repair which may be required, following a design level earthquake, could include bearings, shear keys, and re-positioning of the bridge. While the bridge can be opened to traffic following the immediate repair tasks, follow-up repairs to replace damaged bearings and connections may be required.

Table 1. Expected Seismic Performance

Abutments	• Essentially elastic pile response • Elastic behavior of the connection between the superstructure and the abutment.
Shear Keys	• Pinned connection at bent, may require bearing replacement or repositioning • Shear keys are designed to transfer longitudinal and transverse seismic lateral loads at the bent, but still may show signs of cracking after the design event.
Columns	• R = 4, cracking and spalling. Repair by epoxy injection
Foundations	• Pile Foundations at pile caps will remain essentially elastic.
Joints	• Moderate repair at the roadway-bridge approach transition due to seismic movement and settlement.

If liquefaction remedial measures are not undertaken, then the structure is not expected to survive the design event and collapse is possible. Severe damage would be expected to the piles below grade. The superstructure to abutment fixed connection would be severely damaged and, due to the opposing rotations that the superstructure and abutment may undergo, the damage at this location may result in catastrophic collapse of the superstructure.

For Bridge 37, according to the geotechnical analysis, the soil layer with high potential for liquefaction layer is approximately 4m to 7m thick. Top of the liquefied layer is 8m to 12m below top of embankment at abutments, and is at surface of cutting at Bent 2 (see bridge elevation with liquefied layer). The footing is in the liquefied layer. The postulated lateral spread ranges from 1.4m to 6.7m.

Maintaining integrity of the bridge under lateral spread of several meters is very questionable. Due to proximity of the liquefied layer to the toe of the embankments the stability of the slopes supporting the abutments are likely to be impaired. Slope failures in conjunction with the lateral spread (gross movement of the bridge) could put excessive distortion (rotation and/or settlement) on the piles and abutment/girder connections. Failure of the connection could cause catastrophic collapse of the bridge. At the footing of Bent 2 the postulated lateral spread, if unimpeded, would likely cause the pile/footing connections to fail, in which case the imposed displacements could cause severe damage to the column and/or severe rotation of the footing.
To further amplify the column damage, the difference in stiffness between the bent and the abutments would result in the column being forced to undergo displacements well in excess of the expected ultimate displacement capacity. These displacements would also be permanent, resulting in large secondary moments and shears (from P-Delta effect) being applied to the column. The combination of these events could precipitate collapse of the bridge.

SECTION C6 – SEISMIC DESIGN CRITERIA COMMENTARY

C 6.1 INTRODUCTION

Utah Department of Transportation has developed general procedures/criteria for seismic evaluation and design of highway bridges, based on the nationally published MCEER/ATC-49 guidelines. This commentary document accompanies the seismic design criteria, and expands/clarifies the seismic requirements as needed. As noted in the design criteria, in absence of specific UDOT requirements, MCEER guidelines govern the seismic design. References are included where applicable.

C 6.1.2 BACKGROUND (HISTORY OF SEISMIC DESIGN OF HIGHWAY BRIDGES IN UTAH)

Utah is quite diverse when evaluating seismic risk. Design PGA varies from a maximum of >0.8g along the Wasatch Front to <0.1g in the eastern and southeastern regions of the state. In addition, the greatest seismic risk coincides with the most heavily populated areas of the state.

In 1992, UDOT Structures Division began to require that all highway bridges be designed for a major seismic event. This change coincided with the adoption of the Division I-A Seismic Design requirements into the AASHTO *Standard Specifications for Highway Bridges*. At that time, designers recognized that the potential for a major seismic event along the Wasatch Front during the design life of a bridge (75 years) significantly exceeds the earthquake magnitudes specified in the Division I-A seismic design requirements.

Seismologists estimate that the average frequency of occurrence for seismic events in the Wasatch Front Region is 2 years for Magnitude ≥ 4.0, 10 years for Magnitude ≥ 5.0, 20 years for Magnitude ≥ 5.5, 50 years for Magnitude ≥ 6.0, and 350 years for the MCE. In addition, though the average repeat time for a large earthquake on the Salt Lake City segment of the Wasatch Fault is approximately 1300 years, it has been almost 1300 years since the last one occurred. Therefore, it is likely that a large earthquake will occur during the expected 75-year life of a new bridge. The probabilistic design earthquake in the AASHTO Division 1-A (10% PE in 50 years) does not reflect this fact. For this reason, it was determined that the 2% probability of exceedence (PE) in 50 years more closely represents the maximum earthquake that is expected to occur during the service life of new bridges. Seismic design of bridges was in accordance with the AASHTO *Standard Specifications for Highway Bridges*, Division 1-A.

Since that time, all bridges and retaining walls adjacent to bridges have been designed for a seismic event that represents 2% PE in 50 years. Through the years, seismic experts and the MCEER / ATC 49 bridge design specifications have validated this decision.

UDOT recently adopted the MCEER / ATC 49: *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges* as the governing design specification for all UDOT-designed highway bridges. This state-of-the-art seismic design specification was prepared for inclusion in the AASHTO *LRFD Bridge Design Specifications* but has not yet been adopted by the AASHTO Subcommittee on Bridges and Structures. UDOT Structures believes that it is a more appropriate design approach than that contained in current AASHTO design specifications.

C 6.2 GENERAL REQUIREMENTS

Utah Department of Transportation (UDOT) requires a performance-based design approach for the seismic design of the highway bridges in the state. Realizing the recent development in the field of seismic analysis and design, variability of seismic hazard in the country, and limited resources to guide designers in details of the performance-based seismic design approach, UDOT has hereby adopted MCEER/ATC 49 document titled "Recommended LRFD guidelines for the seismic design of highway bridges", issued in 2003 as the governing design basis for the seismic design of highway bridges in the State.

While UDOT has over the past several years attempted to maintain application of performance-based design principles through agency memorandum, and project specific criteria, the recent effort by MCEER and ATC to compile the pending NCHRP 12-49 document into a design guideline is believed to provide UDOT with an efficient and technically sound design guideline that can be used throughout the state.

While the MCEER/ATC 49 document provides a wide range of guidelines, UDOT via this document has amended, eliminated, or revised the criteria as needed to best apply the technical principles to achieve UDOT's objectives on seismic performance of highway bridges.

C 6.2.2 Seismic Performance Objectives

Performance objectives for the seismic design of bridges are based on the following principles:

- Dual level seismic hazard and performance objective matrix allowing for balancing Life Safety (minimum required performance requirement, also referred to as no collapse), serviceability after the event, and cost.
- All bridges are required to sustain stability to provide Life Safety performance after the Maximum Considered Earthquake event.
- Operational (Serviceability) levels may further be refined for specific projects as immediate service, service after minor repair (48 hours), and service after longer duration repair, subject to UDOT approval. Unless otherwise specified by UDOT, service level for response to Expected Earthquake events shall be Operational after immediate inspection/minor repairs which can be carried out within 48 hours.
- Geometric constraints noted in Table C3.2-1 of MCEER are considered as qualitative guidelines on geometric constraints which affect service after an event. As noted in the table, values shown illustrate typical serviceability "orders of magnitude". Reasonable variations of these values may be allowed for specific projects, subject to UDOT approval, provided the designer can illustrate practical minor repair measures which can provide for serviceable conditions during the allotted repair time span for the project.

C 6.2.3 Earthquake Resisting Systems (ERS) / Earthquake Resisting Elements (ERE)

Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to fulfill the concept should be accomplished during the conceptual design phase, or the situation and layout phase of a project, when design alternatives are evaluated.

Seismic performance is typically better in systems with regular configurations and evenly distributed stiffness and strength. Thus, typical geometric configuration constraints, such as skew, unequal pier heights, and sharp curves, may conflict with seismic design goals. For this reason, it is advisable to resolve potential conflicts between configuration and seismic performance early in the design effort. For example, resolution may lead to decreased skew angles at the expense of longer end spans. The resulting trade-off between performance and cost should be evaluated in the situation and layout phase when design alternatives are viable from a practical viewpoint.

The designer must consider expected damage levels and serviceability requirements when selecting the ERS/ERE for a bridge. Some ERS/ERE are acceptable for Life Safety performance objective, but may not provide the performance necessary for an Operational performance objective. It is the responsibility of the designer to limit the damage to bridge elements to achieve the specified performance objective.

Based on project seismic performance objectives, the designer can establish global seismic design strategies by using specific permitted Earthquake Resisting Systems (ERS) which will provide for a defined lateral load path and meet specified performance goals.

C 6.2.3.1 <u>Permissible Earthquake Resisting Systems (ERS) / Earthquake Resisting Elements (ERE)</u>

Per Section C3.3.1 of MCEER, the ERS's/ERE's described in the following section are permitted to be used for all UDOT projects. Note that these ERS/ERE's can be used to meet Life Safety, Repairable and Operational performance objectives, under MCE or EE events, so long as their limit-states meet the specified performance criteria (at local level as well as global level).

- Limited ductile plastic hinging of flexure members
- Inelastic response of ductile steel bracing
- Fused components (sacrificial elements such as shear keys)
- Bridge bearings (either post failure sliding of typical bearings, or isolation devices)
- Soil mobilization behind abutments
- Limited foundation rocking of spread footings

<u>Limited ductile plastic hinging of flexure members</u>

- Limit plastic hinging to meet plastic rotation and material strain limits, as specified, for performance objectives.
- Typical components include well-confined/detailed columns and steel columns at fixed ends (inspectable).
- Use the overstrength capacity of ductile members as design loads for capacity protected components.

- In limited cases, and subject to UDOT approval, plastic hinging of ductile piling components below walls or pile extensions are permitted. Note that pile extension with smaller strength column portion (diameter or steel strength) is preferred as they force plastic hinging to inspectable region above ground. [Not clear]
- Columns with architectural flares require a nominal isolation gap at the top of the column to prohibit engagement of the architectural segment under maximum column seismic response and to limit the "weak-link" column plastic moment strength.
- Plastic hinging in bent/pier caps and superstructure girders is not permitted.

Inelastic response of ductile steel bracing

- Axial tensile yielding and compression buckling of steel concentric braced frames are permitted, provided specified steel strain limits are met, and the capacity of vertical load support systems meet design requirements.
- Similar behavior and requirements may be allowed for Chevron, K, and Eccentric braces, subject to specific approval by UDOT on a project-by-project basis.
- Design all connections as capacity protected components.
- Use the overstrength capacity of ductile members as design loads for capacity protected components.

Soil mobilization behind abutments

- Soil mobilization behind abutment walls (with or without breaking of the backwall) is permitted, provided performance objectives (Life Safety, Repairable, Operational) are met.
- Passive mobilization of soil behind abutments is allowed. Passive resistance shall consider reaching soil stress limits (determined per site) at displacement level corresponding to 2% of wall height, beyond which no additional resistance shall be assumed.
- When soil mobilization is allowed only after failure of back wall (i.e. fuse behavior), abutment foundations shall resist the ultimate capacity of the wall, in its governing mode of failure, using probable material properties.

Spread footing rocking

- Limited spread footing rocking behavior may be allowed, provided detail soil stiffness and strength analyses are conducted, and post earthquake settlements are accounted for and considered in meeting performance requirements.

C 6.2.3.2 Potentially Permissible Earthquake Resisting Systems / Earthquake Resisting Elements (Requiring UDOT Approval)

The following component performance can potentially be used as Earthquake Resisting Systems / Earthquake Resisting Elements provided the designer can illustrate their effectiveness in meeting project performance objectives, subject to UDOT's approval.

Fused components

- Fusing of shear keys at abutments are permitted, provided adequate seat width is ensured.
- Design abutment foundations to resist the ultimate capacity of shear keys in their governing mode of failure using probable material properties.
- Fusing of shear keys at bents/piers are permitted, provided seat width and restraining devices prohibit unseating of the superstructure.
- In addition to all other design requirements, check in the local push-over analysis protected bent/pier components for the applied horizontal shear force corresponding to the ultimate capacity of shear keys in their governing mode of failure. Use probable material properties.

<u>Bridge bearings</u>

- Isolation bearings are permitted components to reduce seismic lateral loads, provided they meet non-seismic design load requirements and show adequate displacement capacity for estimated demands.
- Typical bridge bearings (elastomeric bearings, pot bearings, sliding bearings) can also be used as fused elements, allowing for "controlled" sliding of the interfaces provided interfaces are designed for sliding behavior, adequate seat width and restraining devices prohibit unseating of the superstructure, and representative sliding friction is accounted for in the design.
- In addition to all design requirements, check protected bent/pier components (local push-over analysis) for the upper bound horizontal shear force transmitted through the isolation devices/fused bearings, also accounting for ultimate capacity of any shear keys used, using probable material properties.
- Fused/sliding behavior is not permitted for high profile bearings with the potential for toppling while responding to seismic loads .

<u>Minor yielding of piles</u>

- Generally, steel piles are among protected components, since they are not easily accessible for inspection and/or repair. As such, their design strengths are required to meet column base overstrength plastic strengths. In pile group cases where minor inelastic response of the individual piles (well within inelastic ductility capacity of the section) can result in considerable economical saving, subject to UDOT determination, the strategy may be permitted. Note that while minor yielding may be allowed for individual piles, the pile group response shall remain essentially elastic.

As noted in MCEER/ATC-49 Section 1.3, it is not the objective to discourage the use of systems that require owner approval. Instead, such systems may be used but additional design/analysis effort and consensus between the designer and owner are required to implement such systems.

The decision as to whether to accept Potentially Permissible ERS's /ERE's (Requiring UDOT Approval) will depend on many factors. In some cases, the geometric constraints of a bridge site together with economic concerns may influence the design such that the use of Potentially Permissible ERS's /ERE's (Requiring UDOT Approval) is the best (overall) solution. As noted in Section 2.2.4 of Caltrans SDC, pile shafts designed to remain completely elastic will increase foundation costs as compared to pile shafts which are designed to allow limited plasticity with in the shaft.

C 6.2.4 Seismic Hazard, Design and Analysis Procedures Design Requirements

The criteria require that all bridges are designed to meet the requirements of SDAP C, D, or E per MCEER definitions. Essentially all bridge designs require performance-based idealization of the structure, subject to the specified acceleration spectra under the specified load combinations, and be designed for either modified elastic design, or modified elastic designs with inelastic displacement capacity check, and capacity-protected design procedures. Per MCEER descriptions and project-specific performance objectives, seismic design and analysis procedure and design requirements are based on:

- Seismic Design and Analysis Procedures (SDAP) C, D, or E
- Seismic Design Requirements (SDR) 4, or 6

Single span bridges with seat type abutments that permit the superstructure to move in both longitudinal and transverse directions over the seat, shall be evaluated per requirements of MCEER Section 4.1, and meet design requirements of MCEER Sections 8.2.5 and 8.3.2, with no additional seismic analysis requirements.

Single span bridges with integral abutments or with seat type abutments but restrained against longitudinal or transverse directions, shall use SDAP C method, using single mode analysis approach.

C 6.2.4.1 SDAP C - Capacity Spectrum Design Method

Regular multi-span bridges with predominantly governed by single mode of response in longitudinal and transverse directions, including single span bridges with integral abutments shall be evaluated using SDAP C procedure.

SDAP C requirements shall meet Section 4.4 requirements of MCEER, as revised herein.

The process uses a single-degree-of-freedom demand evaluation, and requires successive design and check of the columns flexure strength as the key ERS. Columns shall first be designed for service loading, checked and revised for EE, and checked and revised for MCE. Components connected to the column fixed ends, shall be protected components, elastically resisting column plastic hinging forces and moments.

All adjoining component strengths shall be checked to meet capacity design requirements of Section 4.8 in MCEER.

Key Steps for SDAP C include:

Step 2 - Determine effective substructure force-displacement relations by adding response at each substructure to their plastic/yield state, and limit force to V_n (nominal shear) at that support. It is suggested to take 130% of smallest pier yielding event as effective yield displacement $\Delta_{y.}$ Note that Fv and S1 correspond to EE event.

Step 3 – Requires iterations to ensure effective response to EE event remains nominally elastic, by increasing substructure strength/participation to satisfy equation C4.4-2.

Step 4 – Equation C4.4-3 estimates displacement demands for MCE event, based on Step 3 iteration design. Equation C4.4-4 arrives at plastic rotation demands associated with the displacement demands. θ_p shall meet requirements of section 8.7.9 or 8.8.6, as applicable and per performance objectives. Design shall be modified if plastic rotation or strain-based moment curvature analyses are not satisfied.

It should be noted that equation C4.4-4 considers a fixed-pin column condition with height H and base plastic rotation θ_p. For fixed-fixed columns sum of plastic rotations at top and bottom of the columns shall be considered by applying the respective rotations to column portions to contra-flexure points (approximately half of the column height).

Iterative process will then be carried out to strengthen the columns to levels where plastic rotations and P-Δ effects are satisfied.

C 6.2.4.2 SDAP D – (Modified) Elastic Response Spectrum Method

SDAP D requirements shall meet Section 4.5 requirements of MCEER, as revised herein.

The process uses a uniform load or multi-mode response spectrum demand evaluation. Both unreduced elastic forces and displacements are determined for both EE and MCE events.

Demands are modified per specified Response Modification Factors in MCEER, and used to check or revise design strengths. Column shear demands shall be column plastic hinging shear. Columns are the permitted ERS. All adjoining component strengths shall be checked to meet capacity design requirements of Section 4.8 in MCEER.

C 6.2.4.3 SDAP E - (Modified) Elastic Response Spectrum Method with Displacement Capacity Check

SDAP E requirements shall meet Section 4.6 requirements of MCEER, as revised herein.

The process is carried out in two steps. First SDAP D procedure is followed, using higher Response Modification (Reduction) factors. Then stand-alone push-over analyses of substructure systems will be carried out using nonlinear analyses, to the limits of displacement demands established from SDAP procedure, and checked against displacement capacities per plastic rotation or strain limitations governed by performance objectives. All adjoining component strengths shall be checked to meet capacity design requirements of Section 4.8 in MCEER.

C 6.3 LOADS AND COMBINATIONS

C 6.3.1 Load Components

C 6.3.1.2 Live Load

The use of a Live Load factor of 0.25 is based upon recommendations in *NCHRP Report 489: Design of Highway Bridges for Extreme Events.*

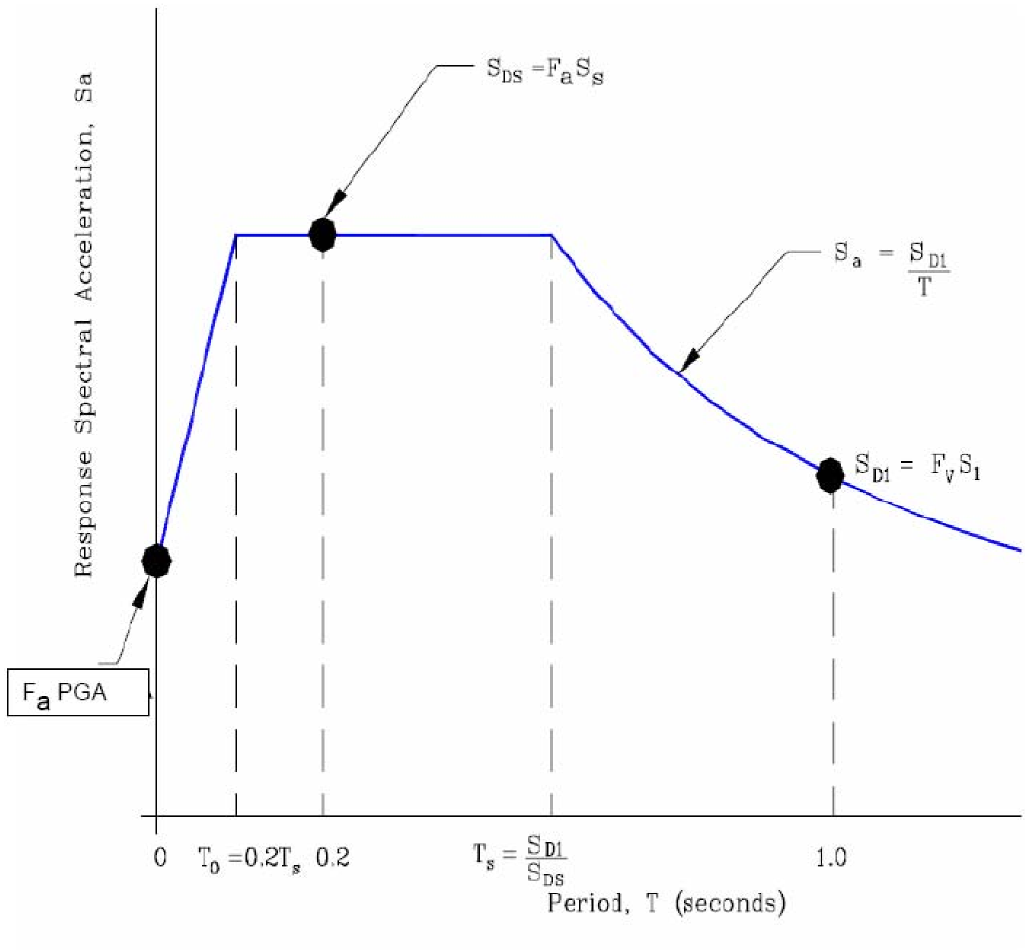
C 6.3.3 Ground Motion

C 6.3.3.1 Design Spectra Based on General Procedure

The two-point method (using S_s and S_1) used in MCEER assumes that the Peak Ground Acceleration (i.e., the spectral acceleration at zero-second period, T=0) can be estimated as PGA= $0.4S_s$ instead of using the actual PGA value obtained from the USGS seismic hazard analysis results. Using PGA=$0.4S_s$ may potentially introduce inaccurate representation of PGA value in some regions. Particularly because PGA is an important parameter in liquefaction evaluation, the on-going NCHRP 12-70 study and NCHRP 20-07 (Task 193) have recommended to define design response spectra using the three-point method (PGA, S_s and S_1) to be consistent with the actual PGA values obtained from the USGS seismic hazard study.

By using the three-point method the design spectral acceleration, S_a, for periods less than or equal to T_0, shall be defined as follows (See Figure below):

$$S_a = (S_{DS} - F_a PGA)\,(T/T_0) + F_a PGA$$



C 6.3.3.2 Site Effects

It is recognized that field-measured shear wave traveling velocity profile is the most appropriate parameter in defining the stiffness profile of the soil overburden at a site and thus the “Site Class” category. In the absence of the shear wave traveling velocity data, empirical correlation procedure has also been used as an approximate method in estimating the shear wave velocity profile as a function of SPT-N values.

For example, for cohesive soils (e.g., clay and silt) the following equation can be used:

$$v_s = 280\,(N_{60})^{0.341}$$

In the equation above v_s is the shear wave velocity in ft/sec, and N_{60} is the SPT-N values obtained using standard 60% hammer energy system.

For granular soils such as sand and gravel the following two equations can be used to correlate the SPT-N values with the shear wave traveling velocity:

$$v_s = (G_{max}/\rho)^{0.5}$$

$$G_{max} = 20{,}000\ (N_{1,60})^{0.333}\ (\sigma'_m)^{0.5}$$

In the equations above v_s is the shear wave velocity in ft/sec, $N_{1,60}$ is the SPT-N values obtained using standard 60% hammer energy system and normalized to an effective overburden pressure (σ'_v) of 1.0 tsf, G_{max} is the small-strain or initial shear modulus of the soil, σ'_m is the effective mean confining pressure of the soil, and ρ is the density of the soil (ρ = total unit weight in pcf divided by 32.2, the gravity acceleration).

Whenever possible, empirical correlations developed based on local data for similar soil types should be used. The resulting shear wave velocities are then used in MCEER Equation 3.4.2.2-1 to obtain the average shear wave velocity and determine the "Site Class" category.

Using the average N-values or S_u-values following the MCEER Equations 3.4.2.2-2 through 3.4.2.2-4 is not considered a reliable procedure in determining the "Site Class" category. This can be evidenced by the fact that the SPT-N value does not have a linear relationship with the shear wave velocity v_s. Furthermore, results based on MCEER Equations 3.4.2.2-2 through 3.4.2.2-4 can be easily biased toward a softer Site Class (i.e., toward Site Class E) by the presence of only a small number of low SPT-N or S_u values.

C 6.4 ANALYSIS

Performance-based seismic evaluation of bridge structures shall be conducted in accordance with MCEER Section 5 requirements, for SDAP C,D, and E, as amended and revised in Section 2.4 of this document.

This section defines individual component modeling to be used in seismic evaluations (global analytical models, as well as local push-over and component evaluation models). Performance-based idealization of structure components is directly related to expected and permitted performance per MCEER definitions and requirements.

C 6.4.1 General Modeling Requirement

Analytical modeling of the structure components, within global analysis, shall consider expected material properties, as described in Section 6.4.3. Individual component behavior shall be specified in the analysis procedure, using representative “element” idealizations, accounting for geometric offsets of connected components at their representative centers of mass and/or stiffness. Figure C6.4.1-a below schematically shows mass, and stiffness distribution of a two-column bent, using beam elements, rigid links, and springs. Note that rigid link modeling will be required, only if depth of the members such as superstructure and bent cap cause significant eccentricity with respect to the centers of stiffness.

Model discretization shall be refined enough to adequately represent mass and stiffness of the structure, for multi-mode analyses. Typically minimum of 4 elements per columns, and 4 to 6 elements per span represent adequate modeling details for typical structures.

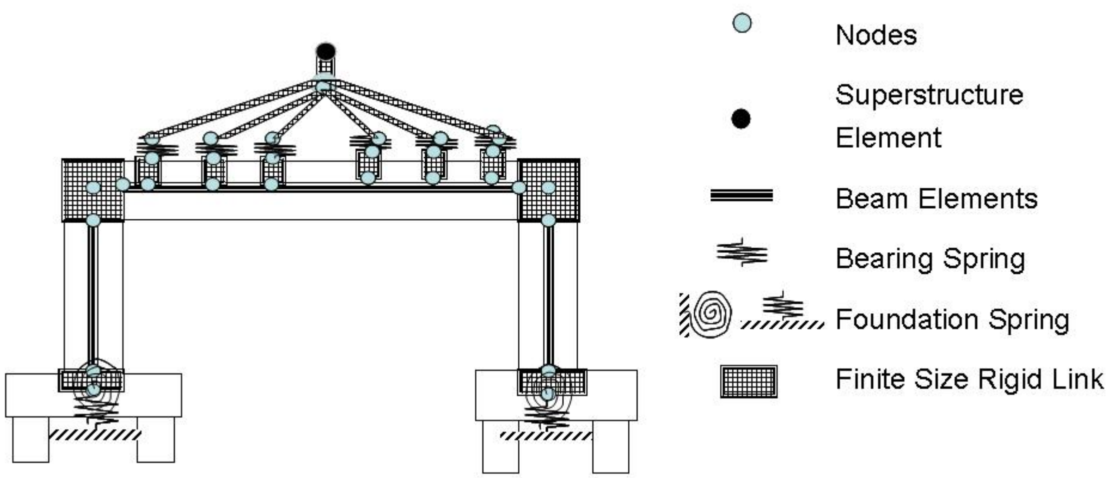


Figure C6.4.1-a - Element Idealization and Geometric Offsets

C 6.4.2 Component Modeling

C 6.4.2.1 Superstructure

Superstructure modeling of the structures shall meet MCEER section 5.3.3.3 requirements, as amended herein. For superstructure components, gross section properties (and not effective cracked properties) shall be used.

Single-line beam elements typically are adequate to represent mass and stiffness interaction of superstructure with sub-structure. It is important to capture skewed end conditions at supports and hinges.

In dynamic analyses, superstructure components are among the protected members. Therefore single-line frame elements with equivalent (i.e. transformed) elastic-linear, gross section properties (area, torsional and moments of inertia) shall be used to idealize superstructure members. Analytical models for truss structures shall explicitly include individual truss member elements, with axial and bending stiffness, concentric at truss joints.

C 6.4.2.2 Substructure Columns

Ductile reinforced concrete columns are permitted in order to limit seismic loads by their ductile (weak-link) behavior, and shall be modeled with their effective linear stiffness. All other adjoining components such as superstructure, bent caps, footings, column end joints are Capacity Protected components, required to remain essentially elastic, and therefore shall be modeled/idealized with their equivalent gross/elastic section and geometric properties.

Typically, other than ductile columns in solid reinforced concrete pier columns, substructures are modeled with their gross section properties, and are expected to resist seismic demands within the elastic range of response.

For Capacity Design of protected components, column plastic and overstrength moments shall be evaluated and used, under the associated axial loads. Alternatively, section moment-curvature analysis required for displacement check analysis procedure can be used to establish plastic and overstrength moments. Overstrength factors shall meet MCEER Section 4.8.1 requirements.

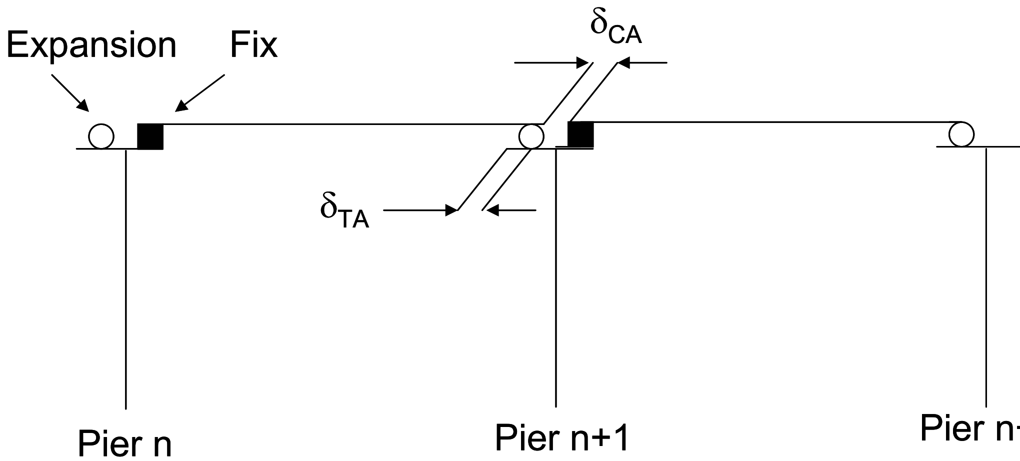
C 6.4.2.3 Expansion Joints

When adjacent structures can have stiffness and/or mass dynamic interaction with the subject structure, effective concentrated stiffness and/or mass in the corresponding degrees of freedom shall be represented. Alternatively Boundary Span(s) can be added to the analyzed segment to represent mass and stiffness interactions at expansion joints.

Terminal or intermediate discontinues shall adequately account for “opening” and “closure” behavior of the expansion joints. This is typically captured by a bounded approach of all open (tension) and all closed (compression) models.

The anticipated joint opening and closure, and their influence/consequence on structure seismic performance must be accounted for in developing representative “Tension” model for joint opening and “Compression” model for joint closure (where necessary).

Joint opening demand shall be limited to allowable δ_{TA} considering seat width. If compression impact is shown to be permitted, allowable joint closure separation (δ_{CA}) can be assumed as joint gap dimension to represent (zero) 'locked-frame' conditions in compression model.



6.4.2.5-a - Expansion Joint opening and closure behavior

C 6.4.2.4 Sacrificial (Fused) Bearings and Shear Keys

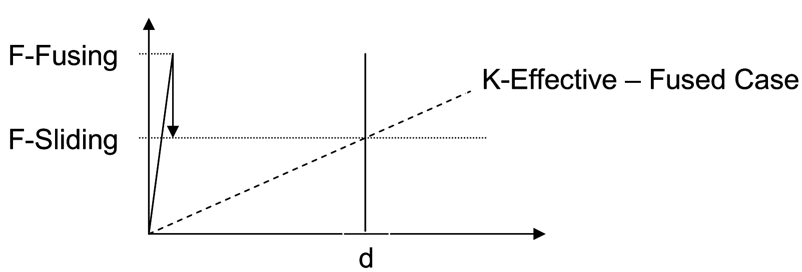
Constraints provided by bearing assemblies, when it can be shown to have repairable consequence upon failure meeting project performance objectives, can be allowed to fail to lead to “Fusing” behavior and thereby protect components from higher loads. This will require that the substructure can be shown to have adequate strength to resist the ultimate capacity of bearing assembly (i.e. bearing assembly ultimate failure force shall be considered in local push-over analysis of substructure components). If fusing of bearing assemblies is used in the evaluation, the corresponding constraints shall be modeled with equivalent linear spring stiffness, reflecting post-failure behavior of the assembly.

Analytical model shall use effective stiffness of the assembly, corresponding to the residual force transfer across the failed assembly interface with substructure, as the upper bound force. For bearings and shear keys, friction-sliding of the superstructure over the seat shall first be determined. Typical steel and concrete friction interfaces are in the range of 0.3 (low for upper bound displacement) and 0.5 (high for upper bound force).

Post-fusing behavior heavily depends on details of assembly connection, plane of failure, and sliding surfaces. When it can be shown that failure will result in dependable sliding interface, the following coefficients of friction μ_f can be used:

$\mu_{f\ min}$ = 0.3 ; use to establish upper bound displacement

$\mu_{f\ max}$ = 0.5; use to establish upper bound force transfer



F-Fusing = Ultimate assembly failure force
F-Sliding = Maximum residual fusing force at interface
(Typical friction coefficients 0.3 to 0.5)
K-Effective = Effective post-fusing linear stiffness

Figure 6.4.2.4-a - Post-Fusing stiffness of Bearing Assembly

C 6.4.2.5 Mass Distribution

Concentrated mass effects (when significant) shall be placed at the center of mass of the components, accounting for geometric offsets using finite size joints, rigid elements, and/or master-slave nodal relationships. Typically, when concentrated mass is specified at center-of-mass of component, in conjunction with finite size and/or rigid links which account for eccentricities, only translational mass (and not rotational) is required.

Tributary structure mass shall be placed at the center of mass of each structural element. The exception would be for longitudinal earthquake forces for simply supported spans where the girder mass and superimposed dead load is transferred via shear (i.e. no moment) through shear keys. In this case, the mass shall be applied at the point of shear transfer (about ½ the key height) to prohibit non-existent rotary effects across the interface.

Note that Live Load effects, when applicable, are to only produce additional loads on resisting components, without their mass being active in the dynamic analyses.

C 6.4.2.6 Abutments

When permitted as ERS, the effects of abutment backwall failure and passive soil stiffness at terminal expansion joints shall be considered for frames immediately adjacent to the abutment. Note that abutment stiffness influence will typically diminish within 2 or 3 frames.

Abutment backwall failure can be considered to be local and repairable (when permitted). The backfill passive resistance may be considered using an effective stiffness (Keff) as shown in the following figure. The maximum passive resistance of the back fill shall be based on site-specific soil data, and limited to 8 ft wall height. Full resistance shall be assumed only if displacements reach 2% of wall height. Note that abutment stiffness shall only be considered in compression (i.e. joint closure case).

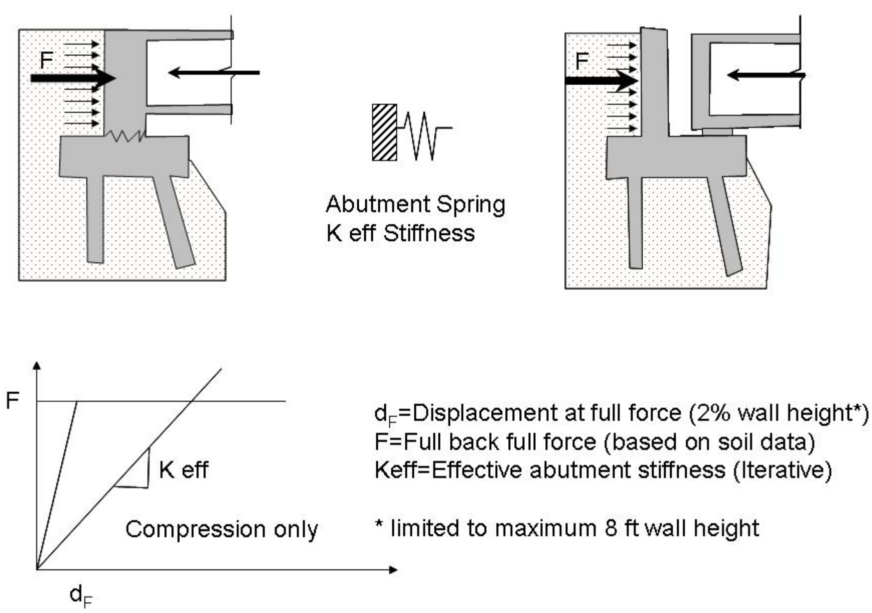


Figure C6.4.2.6-a - Abutment Passive Soil Stiffness Modeling

C 6.4.2.7 Foundations

Representative foundation stiffness for shallow and/or deep piled foundations shall be accounted for within seismic, and/or local push-over analyses. Both structural and soil behavior, and their interaction shall be accounted for.

Geotechnical data shall be based on available site-specific information used for the design.

Typical lateral and rotation stiffness of foundation systems are nonlinear. Analyses modeling of the foundation effects shall be based on equivalent linear stiffness springs. Effective linear stiffness for the foundation effects shall be established iteratively, to correspond to final calculated (converged) demand levels.

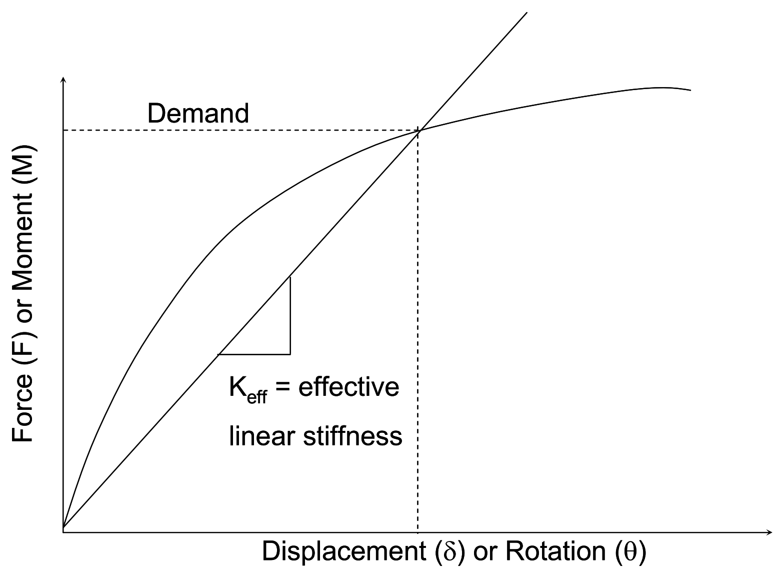


Figure 6.4.2.7-a - Nonlinear Foundation Behavior

C 6.4.3 Material Modeling

Section 4.8.1 of MCEER defines column plastic simplified moment overstrength factors to be used as upper bound seismic load for the protected components.

In general overstrength factors intend to realize the higher material strength, due to material properties as well as material behavior (i.e. confinement of concrete or strain-hardening of steel). Generally, it is preferred to carry out non-linear strain-based moment-curvature analyses to establish column moment strength at design level strains or curvatures. These analyses typically account for non-linear material behavior, however, probable material properties in excess of minimum design strength values shall be specified.

Following sections briefly discuss probable material properties for “Mild Steel”, “Concrete”, and “Structural Steel”. These are well established material behavior models, which also are used by Caltrans. “Prestressing Steel” properties shall be based on design properties, and when used in concrete superstructure is not allowed to reach post-elastic limits. For material strength, probable material properties shall be used for analytical modeling and/or component capacity evaluations. In all cases, material test data, when available, shall be used instead of estimated material properties. In absence of material test data, expected material properties based on specified design properties, as defined in this section, shall be used in the analyses and evaluations.

C 6.4.3.1 Mild Steel

Generally, for seismic design and evaluation of highway bridges the expected mild steel strength f_{ye} as a function of design strength f_y is represented as:

$$f_{ye} = 1.1 * f_y$$

The steel stress-strain model is generally shown in the following figure. Simplified material models (such as linear, perfectly plastic-parabolic) may be used.

Maximum limits of mild steel strains in reinforced concrete columns, caused by column plastic hinging, shall correspond to the plastic rotation limits specified per MCEER Section 8.8.6, for Life Safety and Operational performance levels. In addition to plastic rotation limits for Life Safety performance, mild steel strain shall be checked and limited to ε_{S-A}, as shown below.

Maximum allowable mild steel strain ε_{S-A} is limited to (reduced) ultimate strain, ε_{su}^R

$$\varepsilon_{S-A} = \varepsilon_{su}^R = 2/3\ \varepsilon_{su}$$

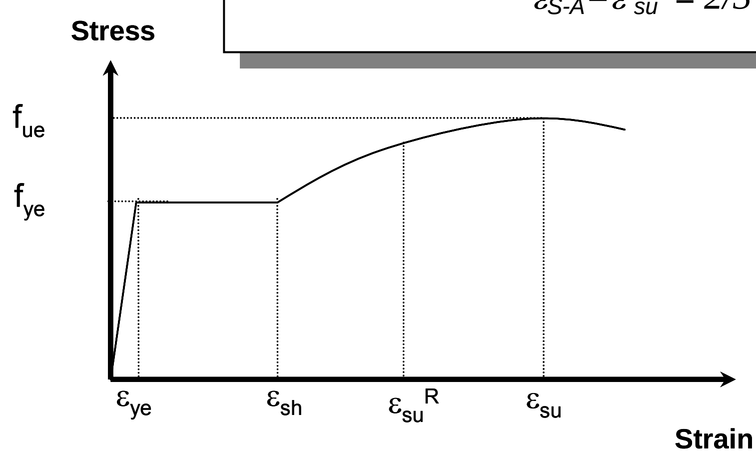


Figure C6.4.3.1-a - Mild Steel Stress-Strain Model

C 6.4.3.2 Concrete

Expected concrete strength (f'_{ce}) shall be used for seismic evaluation and design, instead of design value (f'_c). In lieu of more project specific test results, the following relations can be used:

$$f'_{ce} = 1.3 * f'_c$$

Mander's concrete stress strain model (or equivalent test verified models) for confined and unconfined concrete shall be used, as applicable.

Ultimate concrete strain is a function of concrete confinement, as defined by Mander's model. Maximum limits of confined concrete strains caused by column plastic hinging strains shall correspond to the plastic rotation limits specified per MCEER Section 8.8.6, for Life Safety and Operational performance levels. In addition to plastic rotation limits for Life Safety performance, concrete strain shall be checked and limited to ε_{C-A}, as shown below.

Allowable confined concrete strain ε_{C-A} is limited to ultimate concrete strain ε_{cu} (function of f'_{ce} and confinement)

$$\varepsilon_{C-A} = \varepsilon_{cu}$$

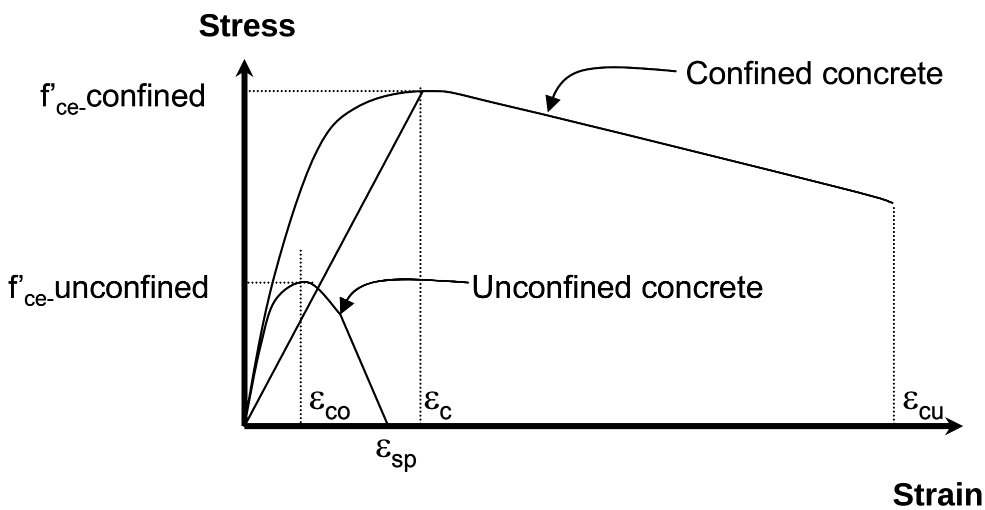


Figure C6.4.3.2-a - Concrete Stress-Strain Model

C 6.4.3.3 Structural Steel

Probable structural steel properties shall be based on material testing when available. In absence of test-verified and approved data, expected material properties using the following relation between expected yield strength fye, and design yield strength fy may be used.

$f_{ye} = 1.1 * f_y$

Simplified and well-accepted steel material properties (such as linear-perfectly plastic-parabolic may be used. Other generally accepted steel material models which have correlated well with test data may also be used, subject to UDOT approval.

Structural steel strain (primarily used in protected superstructure or exceptional substructure components) shall correspond to the plastic rotation limits specified per MCEER Section 8.7.9, for Life Safety and Operational performance levels. In addition to plastic rotation limits for Life Safety performance, structural steel strain shall be checked and limited to $\varepsilon_{SS\text{-}A}$, as shown below.

Allowable structural steel strain $\varepsilon_{SS\text{-}A}$ is limited to strain hardening strain ε_{SH} (at onset of strain hardening).

$$\varepsilon_{SS\text{-}A} = \varepsilon_{SH}$$

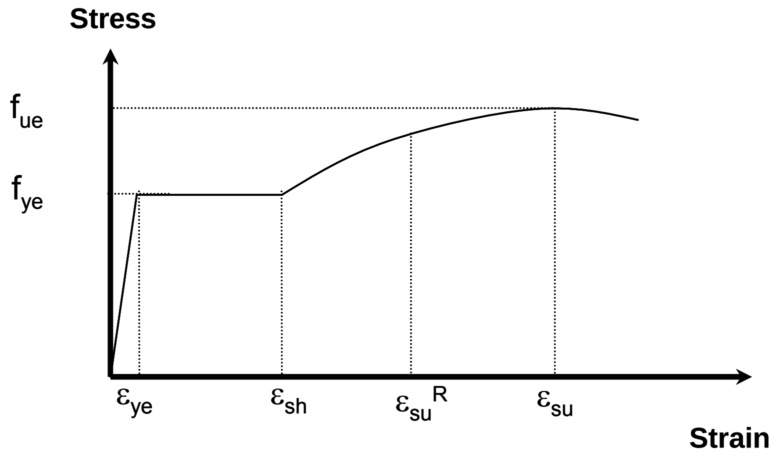


Figure C6.4.3.3-a - Structural Steel Stress-Strain Model

C 6.5 DEMANDS

C 6.5.1 Response Modification Factors

Multi-mode spectral method shall be used to establish demands, for structures with irregular geometry, mass and/or stiffness distribution. The total structure response (i.e. member forces, displacements or relative displacements) shall be computed by combining the respective response quantities from the individual modes by the Complete Quadratic Combination (CQC) method, with sufficient modes to account for minimum participation of 90% of total active mass of the structure for the two horizontal directions. The vertical direction of loading is primarily used for response assessment of the superstructure components.

C 6.5.2 Capacity Design Procedure

Column(s) in the bents are the main elements that are designed for ductile behavior where inelastic deformations will occur in the form of plastic hinges. Additionally, failure of repairable components such as bearing assembly and/or abutment backwall could result to "Fusing" behavior, with similar protection against higher demands on key protected components.

This design methodology, which limits component demands under lateral seismic loads through either column plastic hinging or fusing of permitted components, results in a number of structurally beneficial attributes:

- Limiting the inelastic behavior of the structure to the bents allow for easy access for inspection and repairs after seismic events;
- Reducing the chance of brittle failure in the foundation and bent/superstructure connections, by limiting the load transfer to the weak-link component strength.

Capacity-protected components are designed to have their elastic strength exceed the overstrength capacity of ductile or fuse components.

In addition to column plastic hinging, repairable damage of bearing assemblies which also limit force transfer to adjoined components may be allowed. In these cases, the fusing action shall be checked for failure force leading to the fusing behavior, as well as post-failure behavior, to ensure global and local behavior meet performance objectives (e.g. avoid unseating upon failure and sliding behavior of the bearings).

C 6.5.3 Local Push-Over Analysis Procedure

For SDAP E procedure, local stand-alone push-over models of substructure framing systems shall be developed (accounting for inelastic response of columns.), and analyzed under imposed displacement demands (Step 3 above), per Sections 4.8.1.1, 2, and 3 of MCEER. Analysis shall consider:

- Stand-alone bents, in transverse directions
- Stand-alone frames (Structure between expansion joints), in the longitudinal direction (For the typical simply supported superstructure conditions, stand-alone bent models can be used for longitudinal direction, as well).

- Inelastic column behavior, limiting column moments to an idealized bilinear relationship.

Governing demands for capacity-protected components (all components other than columns) shall be based on forces and moments established from the local stand-alone push-over analyses, or simplified evaluations of limit-state equilibrium where protected components resist permitted ERS strengths (such as column overstrength).

In addition, stand-alone push-over analyses can be used to establish demands for substructure components, under upper bound bearing or shear key fusing force (when permitted). Column plastic hinging is not allowed for a pier where bearing or shear key fusing behavior is used. Therefore push-over analysis for bearing/shear key fusing case will be entirely linear analysis (i.e. no column plastic hinging) to reach ultimate strength of shear key/restraint strengths. Note that in addition to strength evaluations for fusing behavior, displacement response of sliding superstructure shall be checked against Life Safety (i.e. unseating) or Operational (serviceability) limits, as the performance objectives may require.

For components adjoined to column ends where plastic hinging has been reached, plastic hinging of the column in all directions (i.e. governing direction) shall be considered as demand. For components adjoined to column ends where plastic hinging is not reached (i.e. remain elastic), directional combination of column end reactions under longitudinal and transverse push-over analyses shall be considered. The imposed displacements shall correspond to the specified global earthquake directional combination factors 100% and 40% described as part of MCEER specified directional combination factors

Analytical modeling of the stand-alone systems shall properly account for component behavior (linear and/or nonlinear) as well as centers of gravity of applied lateral loads (imposed push) corresponding to location of inertia transfer to sub-structure. In addition, finite size joint offsets, and effective linear stiffness of foundation systems shall be incorporated in the analytical models, accordingly.

Column inelastic response shall accurately be represented (i.e. nonlinear behavior) at each moment resisting end of the column, based on section moment-curvature analysis, or permitted plastic rotations per MCEER criteria. Using nonlinear analysis methods or equivalent step-by-step linear methods, stiffness of a moment resisting end of a column which has reached its inelastic range of response will have to be modified and properly represented.

As noted above, column behavior can be estimated via nonlinear column moment-curvature analysis. This will require the local push-over analysis to be carried out with nonlinear column modeling or piecewise linear modeling which accounts for the nonlinear column behavior. To further simplify column nonlinearity in the local push-over analysis, the following items for typical case of reinforced concrete columns shall be noted:

- Column moment-curvature relation is used to define effective linear stiffness, within elastic range as cracked section property (corresponding to initial yield point for rebar)
- Nonlinear column moment curvature relation can be simplified to elastic-perfectly plastic relationship, corresponding to M_{po} as defined by MCEER criteria. That is to

say, linear from 0 to M_{po}, using cracked stiffness, and then perfectly plastic beyond M_{po}.

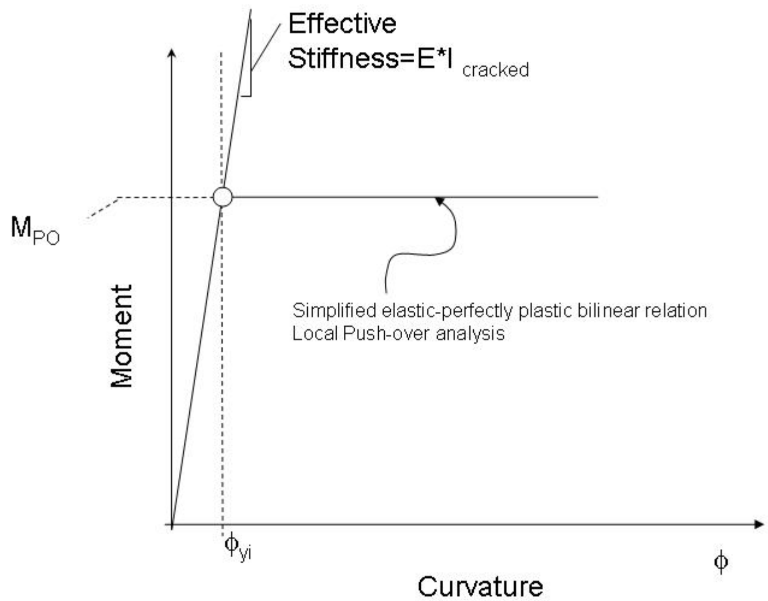


Figure C5.3-a - Simplified Moment-Curvature Relations

- Protected components adjacent to column plastic hinges shall be designed to resist loads causing column moment M_{po}. The analysis can be based on elastic-perfectly plastic relationship between column moment and curvature.
- Column section moment-curvature relationship and Plastic Moment (M_{po}) is a function of axial load. Lateral push-over analyses of single and multiple column bents will have to account for different axial loads in establishing column plastic moment. Multi-column bents and frames shall account for variation of axial load during lateral push, when determining upper bound column M_{po}.

For capacity protection by fusing action of permitted components (i.e. bearing assembly and/or abutment backwall), similar local push-over analysis (at piers or frames) shall be carried out. Local push-over analysis shall account for ultimate fusing force (i.e. failure of permitted bearing assembly) in the form of applied force (in the direction of fusing action). If fusing action is limited to one direction only, local push-over analysis shall consider imposed displacement from global dynamic analysis in the alternate direction, as explained above (for column plastic hinging case).

For fusing option, two important requirements shall be noted:

- In addition to conducting local stand-alone push-over analysis up to fusing force, to establish demands on protected components, global dynamitic analysis shall account for post-fusing behavior (such as sliding of the bearing, or soil compaction behind abutment joints). This check assesses movement behavior of fusing event against required performance objectives. For example bearing shear key fusing shall be limited to avoid unseating for Life Safety performance, while has to be minimal to allow immediate service over the joints for Operational performance requirements.

- At any given pier, either fusing action of permitted bearing assembly, or column plastic hinging is allowed (i.e. not both). Column plastic hinging can be allowed with fused bearings, if additional safety shear keys are provided to limit sliding/fused bearing behavior at higher events. In such dual strategy cases, initial shear key failure aims at limiting force build up and meeting performance objectives for lower EE event, while at higher MCE event the safety shear keys will allow load build up to reach column plastic hinging response.

C 6.6 COMPONENT PERFORMANCE AND DESIGN CRITERIA

C 6.6.1.1 Capacity Protected Components

Two methods of establishing column overstrength plastic moment are recognized by MCEER:

- $M_{po} = \lambda_{mo} M_{n,}$ where λ_{mo} are specifically defined for concrete and steel columns, as well as tubes, piles, and geotechnical design forces. M_n represents nominal strength of the member, however using "expected material properties" as discussed in Section 4.8.1
- Alternatively, M_{po} can be established via strain-based nonlinear column section moment-curvature analyses, using representative steel and concrete material models and expected material properties. Equivalent strain limits for designated performance objectives shall be per UDOT criteria.

C 6.6.1.2 Ductile Components

Requirements of Section 8.2.1 of MCEER shall apply in defining design loads for protected components, as modified herein. Ductility of steel or reinforced concrete columns are limited by specified plastic rotational limits at the plastic hinging ends of fixed columns. The limits vary depending on performance objectives for the project.

C 6.6.1.3 Fuse Components

C 6.6.1.3.1 Superstructure Shear Keys

Capacity design principles need to be applied carefully in the design of shear keys. The designer should determine the order of load application to the system; e.g. columns yield first, then keys engaged, then soil engaged, etc. Following capacity design principles, the shear keys should be designed to engage but not fail the foundation/piles at the abutment or develop plastic shear of the column(s) in the case of a bent.

An overstrength factor of 1.5 is applied for the shear keys to ensure that the structural response is as intended by the designer. If the shear key does not fail as intended, then additional loads may be transferred to the foundations or other critical elements unintentionally.

It is noted that the calculation of V_{nk} per AASHTO LRFD 5.8.4 includes both the cohesion resistance of the engaged concrete surface, cA_{cv}, and the resistance of the interface shear reinforcement, $\mu(A_{vf}\, f_y)$.

C 6.6.2 Superstructure

As long as capacity protection design and detailing is followed for the connection of the superstructure to the substructure, either for bearing or integral connections, the superstructure will be protected from inelastic action resulting from substructure loads.

Vertical acceleration however, must be considered in evaluating the behavior of the superstructure. This can be done through the global response analysis if the structure is expected to behave elastically (including the substructure) or a single mode of vibration, associated with the primary mode of response of the superstructure. Note that this effect is particularly important for near-field sites.

C 6.6.3 Joint and Connections

C 6.6.3.1 Integral Concrete Joints

Joint reinforcements (horizontal and vertical) shall be provided to limit principle core stresses, per MCEER requirements. In all cases, column and or drilled shaft longitudinal and confinement reinforcement shall extend into joint region.

C 6.6.3.2 Steel Member Connection

C 6.6.4 Special Devices

C 6.6.4.1 Typical Bridge Bearings

Failure of bearing assemblies (bearings, shear keys,) can be shown to be limited to repairable damage, and therefore (subject to approval of UDOT) may be acceptable. This behavior also leads to a "Fused" behavior at the interface of the superstructure and substructure, limiting (protecting) attached components.

When such behavior is expected at the bearing assemblies, effective linear stiffness of the components shall be established iteratively, and used in the analyses. The maximum force transfer upon failure depends on design and weak-plane of failure. For friction between steel and/or concrete interfaces, coefficient of friction of 0.3 to 0.5 has typically been used.

Note that although the force limitation of fused behavior (protection of adjacent components) is an advantage of "fused" bearing assembly, the resulting larger displacements at superstructure-substructure interface shall be checked against performance limit states (i.e. available sliding seat width for Life Safety, or minor opening for Operational). Also, note that the protected components shall be evaluated for the "Upper Bound Force Demands", which in the case of "Fused Bearing Assembly" is the ultimate failure load of elements such as shear keys or anchor bolts, prior to fusing.

C 6.6.5 Substructure

C 6.6.5.1.1 Solid Reinforced Concrete Columns

Column section moment-curvature analyses provide:

- Column plastic-strength relations to axial load
- Identify initial rebar-yielding to represent effective/cracked section property
- Determine plastic curvature capacity as a function of steel or concrete strains
- Distribute plastic curvature along plastic hinge zone to determine plastic rotation capacity
- Use plastic rotation capacity at the center of plastic hinge zone, to determine plastic displacement of column.

Figure C6.6.5.1.1-a illustrates typical moment-curvature relations for reinforced concrete columns.

Section moment-curvature results can directly relate allowable steel and concrete strains to allowable curvatures, which when distributed along the length of concentrated plastic hinging (L_P) it will arrive at plastic rotation.

Column curvature distribution, column deformation, and forced displacement relations for a single column bent are shown in the Figure C6.6.5.1.1-b.

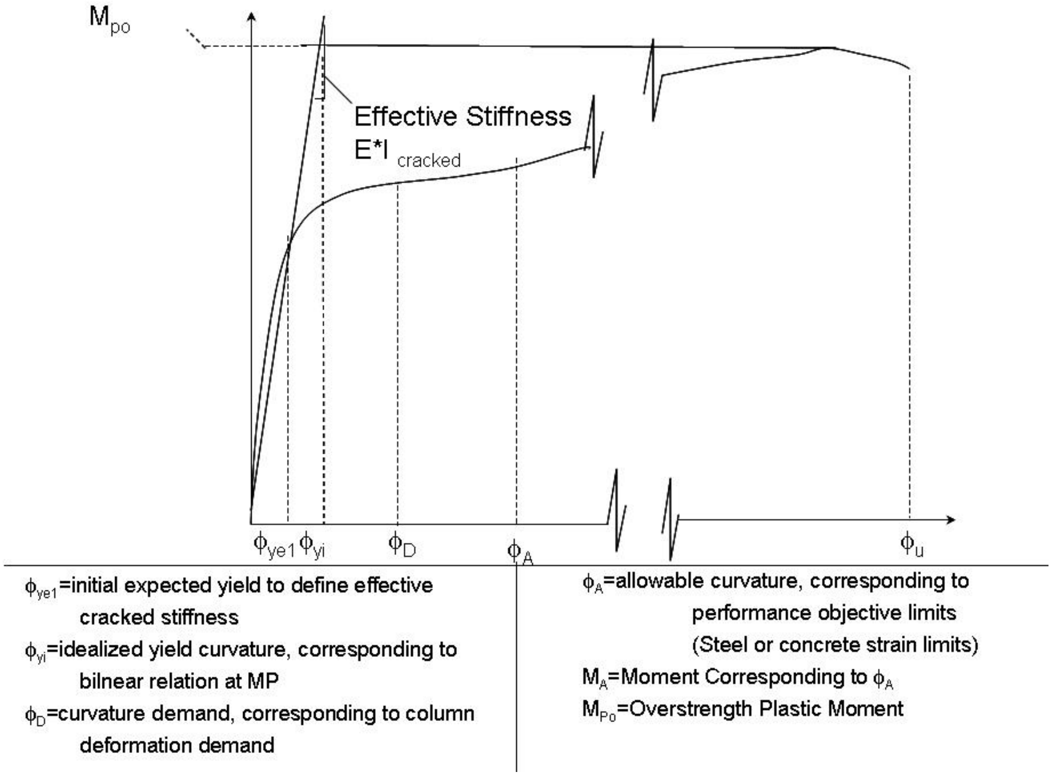


Figure C6.6.5.1.1-a - Column Moment-Curvature Section

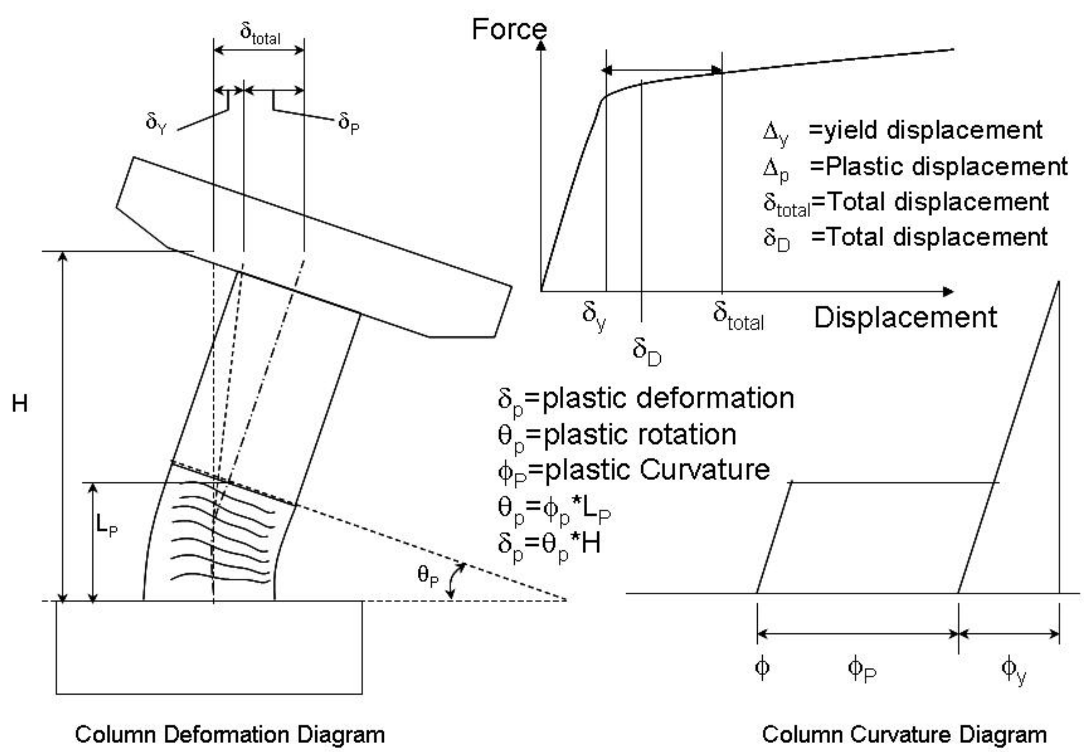


Figure C6.6.5.1.1-b - Idealized Cantilever Column Plastic Hinging Force-Displacement

C 6.7. STRATEGIES AND DETAILS

C 6.7.1.2 Earthquake Resisting Systems (ERS) With Permission (See Section 6.2.3.1)

The design of pile foundations in competent soil can be greatly simplified if we rely on inherent capacity that is not directly incorporated in the foundation assessment. For example, typically pile axial resistance exceeds the designed nominal resistance and axial load redistributes to adjacent piles when an individual pile's geotechnical capacity is exceeded. (Caltrans Seismic Design Criteria (SDC) Section 7.7.1)

C 6.7.3 Foundation Design

In addition to the foundation requirements of MCEER, this criteria makes special reference to the interpretation of how foundation plastic and overstrength loads are to be applied to the foundation and pile system. Ref: Caltrans SEISMIC DESIGN CRITERIA _ DECEMBER 2001 VERSION 1.2, 7.7.1.1 Pile Foundations In Competent Soil.